ENGINEERING NEWS

A JOURNAL OF CIVIL, MECHANICAL, MINING AND ELECTRICAL ENGINEERING

Vol. LI. No. 15. New York, April 14, 1904.

TABLE OF CONTENTS

LEADING ARTICLES: Page

Experience in the Erection of American Viaducts on the Uganda Ry. (illustrated). A. B. Lueder 345
First Impressions of Municipal Work in Great Britain (Editorial Correspondence) 347
A Support for Water and Gas Service Cocks and Boxes (illustrated)..... 348
Annual Meeting of the American Electrochemical Society 348
Practice in Re-Establishing and Monumenting a Railway Center Line (illustrated). Albert I. Frye, M. Am. Soc. C. E..... 351
Cross-Bending Tests on Concrete-Steel Beams (illustrated). M. A. Howe, M. Am. Soc. C. E..... 359
Eight-Roll Plate-Straightening Machine (illustrated 359
Some Recent Tests of the Weight of Crowds. L. J. Johnson, Assoc. M. Am. Soc. C. E..... 360
Current Practice in Apartment House Construction in New York (illustrated).. 361
Some Comments on Building Laws. Gunvald Aus, M. Am. Soc. C. E..... 361

Page

Reinforced Concrete Roof for a Locomotive Roundhouse, Long Island R. R. (illustrated) 363
A Useful Table of Weights of Cast-Iron Pipe and Flanges..... 364
Electric Conduit Construction at Memphis, Tenn. (illustrated). F. G. Proutt.. 364
ENGINEERING NEWS OF THE WEEK..... 351
EDITORIAL COMMENT.—A Series of Reports on Foreign Practice in Municipal Engineering——Stronger Construction for Passenger Car Frames——The Illinois Central Steel Frame Passenger Cars——Suggested Changes in Building Laws——Floor Loads and the Weight of Crowds 352
EDITORIAL.—Structural Engineering in Apartment House Construction 353
LETTERS TO THE EDITOR.—Recording the Location of Sewer House Connections: Edward Mohun, M. Am. Soc. C. E.——The York, Pa., Sewer Specifications Again: Alexander Potter——Elevator Car-Safety Appliances and the Building Code: Edwd. Van Winkle——Horizontal Reinforcement vs. Trussed Reinforcement of Concrete Beams (illustrated): Maurice Goldenberg——Shearing Stresses in Concrete Steel Beams (illustrated): A. L. Johnson and Julius Kahn 354-358

EXPERIENCE IN THE ERECTION OF AMERICAN VIADUCTS ON THE UGANDA RAILWAY.

By A. B. Lueder.*

As the readers of Engineering News will recall, one of the most notable foreign contracts carried off by American engineers during the past few years, was that for the bridge works on the Uganda Ry., built by the British Government in East Africa. The fact that the contract was given to an American firm created no small sensation in England, and was even the subject of discussion in Parliament. The following extract from the Parliamentary proceedings of Dec. 21, 1900, not only makes plain the state of public sentiment there, but also the conditions under which the contract was secured and carried out:

Sir A. Hickman (Wolverhampton, W.) asked the Secretary for the Colonies whether he was aware that orders involving expenditure of many thousands of pounds for bridges for the Uganda Ry. had been given to an American firm; and whether he would take steps to prevent such orders from being given in future without full opportunity being given to bridge builders in the United Kingdom to compete on the same conditions as regards tests, mode of manufacture, inspection, and erection at makers' works, as the American makers were subject to.

Viscount Cranbourne replied: "The answer to the first paragraph is 'Yes.' With regard to the second paragraph, tenders were invited in the United Kingdom on drawings and specifications prepared by Sir A. Rendel & Co., the consulting engineers, under the same conditions as regards tests, etc. One of the American tenders was found to be considerably the lowest in every respect, and was, therefore, accepted. British firms are always given full opportunity to tender for work for this line, and I need not say that Her Majesty's Government are anxious to employ them whenever the terms offered are consistent with the interest of the public service."

Mr. T. Bayley asked the Under Secretary for Foreign Affairs whether an American company had contracted with the Foreign Office to erect 34 bridges on the Uganda Ry., requiring something like eight thousand tons of structural steel; and if so, whether he would state the amount of the contract, and whether any tenders for these bridges were invited from well known British firms.

Viscount Cranbourne: Yes. The American Bridge Co. have contracted for the erection in situ of thirty bridges on the Uganda Ry., containing approximately seven thousand tons of structural steel. The amount of the contract is for £135,000. As the hon. member will have learnt from the answer already given to the hon. member for Wolverhampton, tenders were invited for the contract in the United Kingdom. They were invited by means of advertisement."

Mr. Cremer asked the Government if they would state the names of the British and American firms who tendered for the erection of bridges for the Uganda Ry., the amounts of the various tenders, and if the British firms whose quotations were higher than the American were invited to or afforded an opportunity of revising their tenders.

Viscount Cranbourne: "I shall be glad to give the information privately to the hon. member, or to grant it as an unopposed return if he will move for it. It is not considered fair to the party making the lowest tender to invite other parties to revise their tenders." (Some ironical cheers.)

Sir Alfred Hickman subsequently asked whether steps would be taken to ensure that lists of materials to be used in the Uganda bridges should be made, and that a thorough inspection of the bridges when erected at the makers' works would be made in the same manner as if the bridges were made here. He also wanted to know whether the Under Secretary was aware that, on the average, 20 bridges break down in America every year.

Lord Cranbourne declared that the materials and work on the Uganda viaducts would be inspected in the same way as if they were built in England, and by an English engineer of large experience in ironwork. Inasmuch as the contractors have undertaken the erection of the bridges in situ, he did not think it probable that they would erect the girders in their yard in America in the same way as is customary for bridges in this country, where the conditions are commonplace.

It was the writer's fortune to be placed in charge of the work of erection of these viaducts, and some notes regarding his experiences may be of interest to the readers of Engineering News. The journey to British East Africa was made from New York to London, thence across the Continent to Naples and thence by a steamer of the German East Africa line through the Mediterranean, the Suez Canal, the Red Sea, and the Indian Ocean to Zanzibar. Here a coasting steamer was taken to Mombassa, and the site of the work was found between miles 470 and 540 of the railway connecting Mombassa with Lake Victoria Nyanza.

At the time of the writer's arrival at Mombassa in May, 1901, the Uganda railway had already reached mile 490, passing the sites of the viaducts by means of temporary switchbacks running up one side of the ravine, often on a 4% grade, crossing the ravine a mile or so above the permanent line, and coming back the opposite bank of the ravine, and then by another switchback reversing the train to bring the locomotive at the head again. This extra work was necessary to get material ahead to complete the line, as there was absolutely no means of transport except negro labor. Bullock carts and pack mules were used, but only at great expense, and they could not have moved the bridge material, besides the railway managers could not afford to wait until the bridge work was completed before moving ahead to grade the road and lay rails. Their object was to reach Lake Victoria Nyanza in the shortest possible time, and then come back and put the permanent line in at their leisure.

At one place the Kikuyn Escarpment, some hundreds of thousands of pounds were expended on a temporary line to save about six months time.

The railway is of meter gage. The locomotives are about half American Baldwins and the other half of English make. The "goods cars" are of 10 tons and 20 tons capacity, and are all of steel. The passenger coaches are divided into first, sec-

A Gang of Riveters on Pay-day.

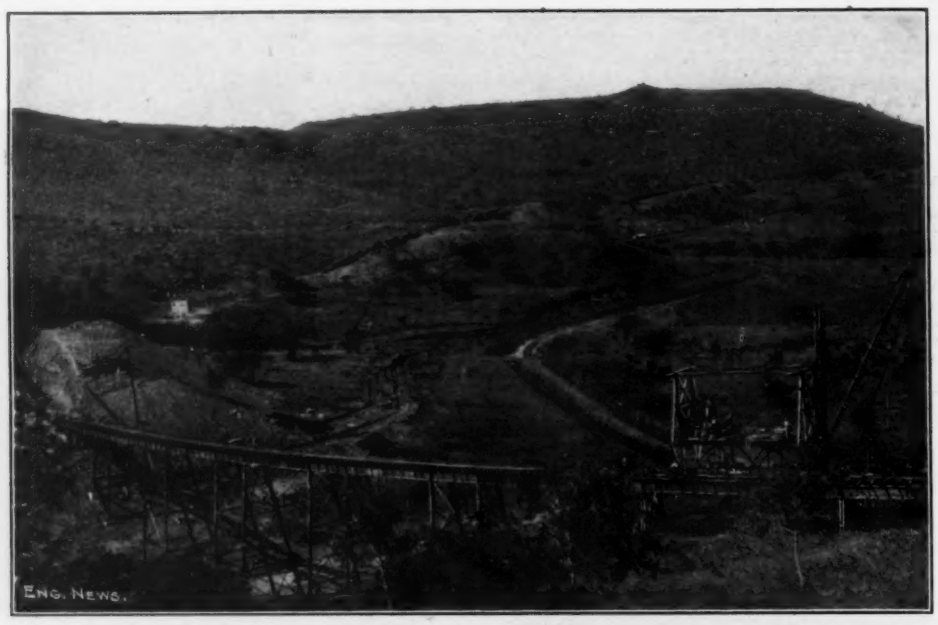
Indian Carpenters at Work.

The Nucleus of a Good Riveting Gang in Sunday Attire.

SAMPLES OF THE LABOR FORCE ON THE UGANDA RAILWAY BRIDGES.

*Engineer Erecting Department, American Bridge Co., Pittsburg, Pa.

ond and third class. They have steel frames and are about 24 ft. long and 6½ ft. wide. The first-class compartments, of which there are two to the car, are supposed to seat six in the daytime and furnish sleeping quarters for three, an extra berth being on one side. The second class has the same size compartment, which is designed for eight in the daytime and four at night. The third class is not limited as to number, and these cars would fill up at times until they looked like a car going to a circus ground.



A TYPICAL VIADUCT ON THE UGANDA RY., BRITISH EAST AFRICA.

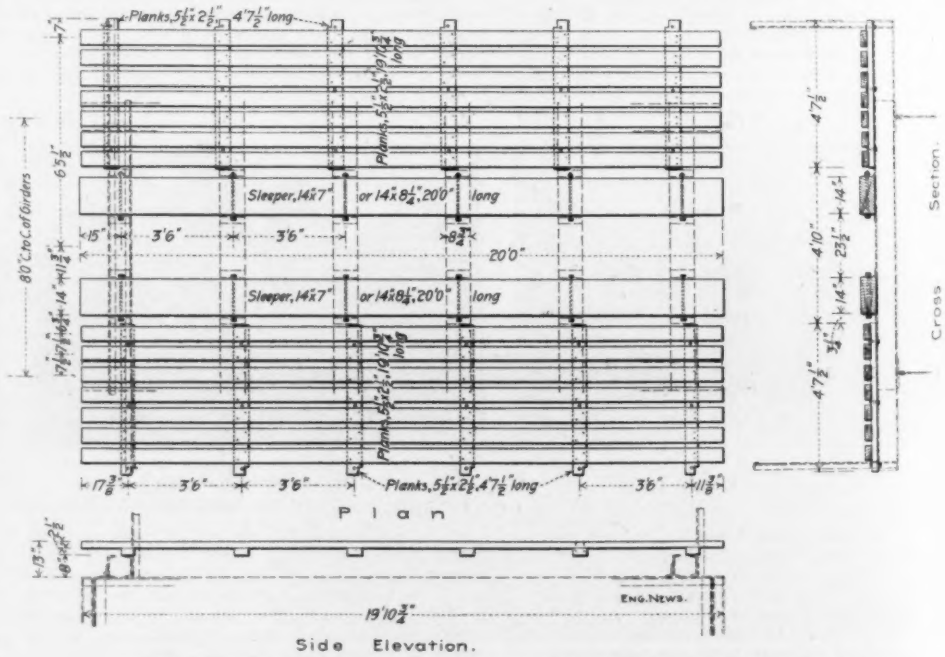
In traveling one always had to carry his own blankets and food, as until the road was fully opened, no accommodations could be had on the route, and it was no uncommon thing to be a week or ten days going 500 miles.

The railway line in itself is a curiosity, and the country through which it passes is probably the most interesting, at the present time, in the world. For the first 300 miles the road steadily rises up to an altitude of 6,500 ft, and then it suddenly drops, in a distance of eight miles, to 5,400 ft. Then it passes over 50 miles of practically level country; after which it begins to rise again and at mile 490 comes to the extreme elevation, 8,300 ft. It now begins to descend and in the next 90 miles drops to practically 4,000 ft., at Victoria Lake.

The country through which it passes is at first quite hilly and difficult; then comes a vast plain, the Toro desert, where the country for 100 miles is covered with thorn bushes, cacti, etc., hard on an engineer's temper. Here water had often to be carried 40 miles on porters' heads for the use of the engineers on survey. Next comes great grassy plains simply swarming with big game. On these plains there were so many lions that the labor force came near to becoming disorganized, as many as 22 men having been killed by these brutes. After the plains came forests at the escarpments, and then rolling plains again. These large forests are mostly of olive and juniper, and other woods of no commercial value beyond fuel. They furnish the fuel for the railway locomotives and for our hoisting engine.

The bridgemen, upon their arrival in Africa about the middle of December, 1901, were immediately shipped from the coast, which is a malarial region, to the site of the viaducts. Here good quarters had been prepared and a large gang of Indians and Africans were waiting for foremen to direct them in their work. As soon as the men had their bearings, squads of these natives and Indians were put in charge of a white man with some special work to do. At first there was considerable trouble in handling the men on account of the language, but the Americans quickly picked up some words and soon had their men under good control.

One sub-foreman had charge of the unloading; another sorted the iron, put as much together as possible in the yard, and then sent it to the foreman of erection on the viaduct as it was needed. As much of the lighter material as possible was distributed by negro labor on the ground as near underneath its final position in the structures as possible. This was a much cheaper method than handling all material overhead through the traveler. All carpenter work and most of the riveting was done by Indians. They are very good carpenters, although slow. Probably an American carpenter could do three times what they would in a day. Their most serious fault was a total inability to read plans. The riveters were mostly men who had been taught in Indian shops, or on viaducts previously built by the British Government. Ninety ⅞-in. rivets or 110 ¾-in. were considered a day's work. To stimulate the work, we tried to get men to work by contract, and did finally persuade them to do so, when they drove as many as 300 ¾-in. rivets a day!



PLAN AND CROSS-SECTION OF FLOOR OF VIADUCTS FOR UGANDA RY.

The Indians mentioned were men indentured to the English Government by the Indian Government for work on the Uganda Ry. for periods of two to three years. Their pay varied anywhere from 12 rupees to 45 rupees ($4 to $15) a month and food, the latter consisting of flour or rice, "dall," a sort of split peas, salt, peppers and clarified butter. A month's rations would cost about $3.25. It was exceedingly cheap labor even if one good white man is equal to three or more Indians.

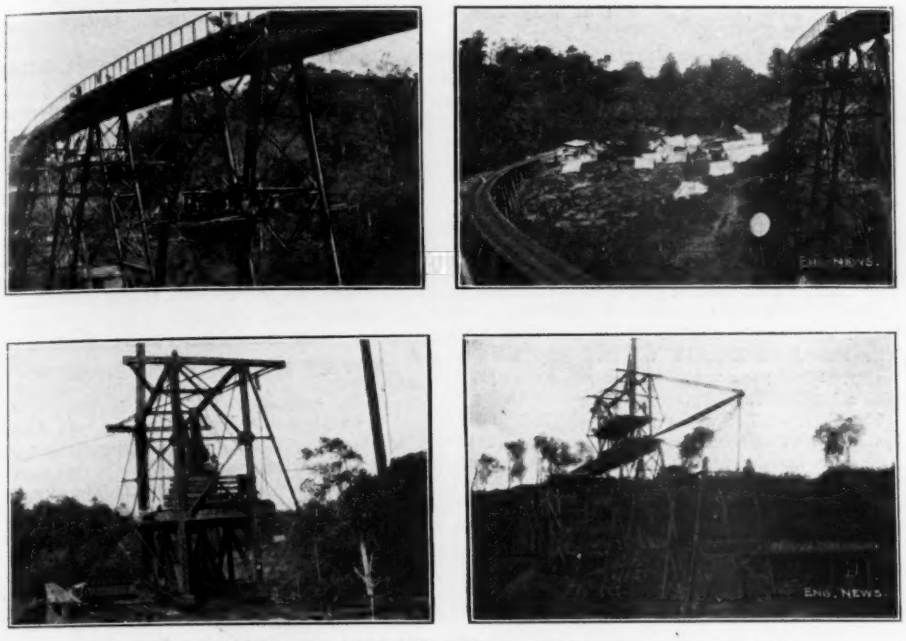
By the natives I mean those negroes native to Africa. The old slave traders have built up a very fair type of negro at the coast, and it was here we engaged most of our natives, these men knowing something of the whiteman's ways and being willing to agree to stay with us for six months or longer. The interior negro is, generally speaking, a useless fellow, with the exception of the tribes just north of Lake Victoria, the Waganda, from whom we had some men, whom we found fairly intelligent, although extremely lazy. The best and most faithful workers came from German territory, the Wanwumwazi, who are, as a rule, good, stalky, strong men who will stand a great amount of really hard work. These men had generally seen service on the great caravans which entered this region for trading purposes and slave capture, or exploration. They were old hands at carrying loads on their heads, but the handling of bridge iron was a new proposition and necessitated a very considerable amount of teaching. A serious fault of these men was that they would get tired of working and wish to go home, about the time they had learned how to do our work, and we would have to procure new labor. These men we hired for 10 rupees, about $3.30 a month and food which cost us about 3 cts. per day, the food consisted entirely of rice, 1½ lbs. a day and a pinch of salt.

In handling these men it was necessary almost to go back to old slave days. They could be punished in no way except by the whip or by fining them. This latter method was undesirable in most cases and not effective, so that it was necessary to hand the culprits over to the police, who carried away your laborer and thus deprived you of a man whose place you could not fill immediately. The only alternatives were to either overlook the offense or act as police, judge, and executioner yourself. This latter plan we found effective in most cases and to be favored by those in authority in the country; in fact, the man in responsible charge of work was made an officer of the law with the power to deal with trivial crimes in cases where the police were not easily accessible.

The accompanying photographs will give the reader a fairly accurate idea of the work, which, of course, was quite light, being for a single-track meter gage road. The spans were of 40-ft. and 20-ft. girders and the bridges varied from 120 to 880 ft. in length and from 15 ft. to 112 ft. in height. The tower bracing consisted of single

angles for the diagonal bracing and four angles laced for struts. The columns were built up of four Z-bars and a plate. The 40-ft. girders were braced by three sets of horizontal bracing and four sets of vertical bracing. The 20-ft. girders had three sets of vertical. Steel ties of 7-in. channels were used, upon which a longitudinal balk of 7 × 15-in. long-leaf yellow pine was laid to carry the rails. This was the weakest point in the bridges, as most of them were built on curves and the balks were too narrow to carry both the guard rail and track rail at the same time and give a good grip to the spikes.

All the material was fabricated at the Pencoyd



VIEWS AT VIADUCT "O" DURING ERECTION.

Plant of the American Bridge Co., at Pencoyd, Pa., and was inspected by an English expert. In order to reduce the chances of the material getting mixed in the field, each separate bridge of each shipment (of which there were three of about 2,000 tons each) was painted a separate color. Each bridge was also given a distinctive number and named a distinctive letter, such as K, L, M, N, etc., and lastly each piece had its own distinctive erection mark. The material was very carefully checked as it was completed and stored in the yard at the shop; it was again checked as it was loaded on the cars, and again as it was put aboard the vessel at New York.

Upon the arrival of the first shipment of material at Mombassa, it was again checked as closely as possible as it was unloaded. The material was hoisted out of the hold of the vessel and placed in lighters by a contractor for an absurdly low price (about 3 cts. a ton), he using the vessel's winches. The material was conveyed by the lighters to the railroad wharf and here unloaded from the lighters by the railway company, using their own cranes. It was then loaded onto trucks and run to a stacking ground. The railway people found it inconvenient with their facilities to sort the material as it was unloaded, so it was thrown off the cars in the yard in hit or miss fashion. After all had been unloaded, the railway company sorted the material by means of the distinctive colors and marks into separate piles, all easy of access for shipment to the site. As soon as the site for unloading near each bridge was determined upon, notice was forwarded to the coast and the material for that bridge was forwarded. As there was no piece weighing more than five tons, it could all be handled easily by the 10-ton traveling crane in the yard. As the material was received at the bridge site, it was carefully checked by the bill of material and notice was forwarded to the coast if any pieces were missing. Four all-steel derricks, with masts 45 ft. long, and 60-ft. booms in 15-ft sections had been shipped with the tools, so that as soon as the yard for each bridge was determined a derrick was erected and the material unloaded. This derrick was then left standing for use by the erecting force and another derrick was brought up for the next site.

The bridge contractor was not concerned with the loading and hauling of this material from the coast, the railway company having agreed to do this, but the bridge contractor did the unloading of the material at the bridge site. To get it there necessitated in many cases the building of temporary spurs from the railway down very steep grades, but as the labor was done by coolies who cost the railway about $6 per month, this was done quite cheaply. We found that our unloading was a very small item of expense, as our unloading force consisted of about 25 native Africans at 10 rupees per month each and food, and one American foreman.

After the erecting force reached the bridge sites, the yard foreman immediately began to sort and assemble the iron. He also had three or four riveting gangs who drove as many rivets as possible in the yard, as we found that rivets driven on the ground by negroes cost very little. As soon as the material was ready in the yard it was run to the traveler foreman on the bridge, who placed it in the structure.

Meanwhile all small pieces of bracing, plates, etc., not riveted in the yard and not weighing over 300 lbs. were carried by negro laborers to a position as near directly under where they would finally rest as possible. As the erection proceeded we found that very considerable time was thus saved, as it did not necessitate so much raising and lowering of the traveler booms to pick up the material.

The all-steel traveler used contained no new features. It was designed for the narrow-gage road and ran on rails placed directly over the girders of the viaduct. The girders were on 9-ft. centers. The hoisting engine (two drums and four spools) was on a platform about 10 ft. from the track level, thus leaving an open space through which to run material.

The traveler was rigged with two all-steel booms, one 60 ft. and one 45 ft. The 60-ft. boom was used in erecting the towers and putting in bracing, while the 45-ft. was used in placing the 40-ft. girders. The handling of this traveler was much the same as may be seen any day on structures in process of erection in the United States. The most striking feature on the work was to see Indians holding bolts in the holes with their bare toes while they put the nuts on with their fingers, or to see an Indian climbing a column grasping the rivets with his toes.

Our camp for the white men consisted of four corrugated-iron bungalows, 40 × 12 ft., built on posts about 4 ft. from the ground, and in most cases the roofs were ceiled. These houses were divided into two rooms with three men to a room. Tables, beds, chairs, etc., were made by the Indians, and as plenty of mattresses, blankets, sheets, towels, etc., had been sent by the bridge company from America, the camp was made very comfortable. Each room was allowed a servant to carry water for washing purposes and to act as table waiter.

The food, with the exception of meats and fresh vegetables, was sent from America, and consisted of everything a camp could need. For meat we were able to get cattle, sheep and chickens; also a very considerable amount of venison was to be had.

The water in the mountains was invariably good, but on account of so many construction camps being near the streams, we considered it advisable that all water should be boiled and that rainwater should be used whenever it was available.

As for sickness, we had an extremely small amount, not more than one would expect in the same time and with the same number of men in a temperate climate. The principal troubles were slight cases of sun fever, some malaria and a few slight cases of dysentery. By taking these cases promptly in hand, the trouble was checked and the work was carried through successfully without the loss of a man.

The Indian and native camps were kept in as sanitary condition as possible, pools of stagnant water were abolished, and the men were made to sleep on beds and not on the bare ground, as they liked to do.

These laborers lived almost entirely in tents, from six to eight Indians or eight to fifteen negroes to a 10-ft. × 12-ft. tent.

The English engineers on this work treated us with uniform courtesy and afforded us all the help that could be expected. Some little delays were caused us in teaching the laborers, or rather in getting labor that could be taught. We were considerably behind the foundation work when we started the erection, but caught up and had to wait upon it, before we had completed the erection. The railway company put in the foundations.

The contract called for the viaducts to be erected complete 28 weeks from the date the foundations were ready. It took just 53 weeks to complete them.

The chief engineer of the railway, Sir George Whitehouse, took occasion in his final letter of acceptance of the work to compliment highly the erecting force for their speed and good workmanship, and for the hearty way in which his suggestions were adopted.

FIRST IMPRESSIONS OF MUNICIPAL WORK IN GREAT BRITAIN.

(Editorial Correspondence.)

Having undertaken a study of municipal activities in Europe, chiefly in Great Britain, it was natural that I should begin in London, which is not only the greatest center of municipal life in the world, but which is also, in large measure, the seat of control of municipal undertakings throughout Great Britain. This control arises (1) through the fact that Parliament and its central administrative agencies for municipal affairs, such as the Local Government Board, are located here; and (2) because here is centered the engineering and allied technical talent of the country.

After spending two very busy weeks in London and vicinity, during which I have conversed with many people and seen quite a variety of municipal work, I now venture to state some general impressions, which may serve as an introduction to the more detailed communications which will follow.

As I have thus far only one decidedly unpleasant first impression to record, and as it was my very first, I will have done with it at the start. My enjoyment of my short railway ride from the Tilbury Docks of the Atlantic Transport Line to London was marred by the advertisements within the right of way of the railroad, and I have scarcely yet become accustomed to the many-col-

ored glowing advertisements covering the exterior of the omnibuses and belittling if not obscuring the comparatively small lettering that gives the route traversed by each omnibus. Doubtless these things are more annoying to strangers than to those accustomed to them, but it is disconcerting to a stranger, cooped up, alone, it may be, in a compartment of an English railway coach, with no trainmen to announce the stations, to have his eye met with "Bovril" in clear, large type, when he is most anxious to know whether or not he has reached a small suburb of London. Neither does "Nestle's Food," in large letters on a bright yellow background at the front of an omnibus, contribute materially to one's peace of mind when trying to select the proper 'bus to reach a given destination.

From 'buses it is natural to turn to the admirable manner in which the enormous street traffic in London is controlled. The rule of the road, conspicuously posted, at crowded points, is, "Keep to the left," and thus there are two main streams of travel. Consequently, the pedestrian, in crossing a street, is not compelled to keep an eye out for traffic in both directions, while at the worst corners, and often in the blocks, are "isles of safety" or raised sections of the streets, midway in its width, or even dividing the crossing into three stages. At crowded street intersections the policemen hold up first one and then another of the four streams of travel, avoiding confusion among the vehicles and giving foot passengers an opportunity to cross with ease. This general system of controlling traffic at street intersections, many readers will note, has been in successful operation for a few months in New York city, but the wonder is that it was not introduced there long ago and that the rule of the road, which would, in America, of course, be "Keep to the right," is not also rigidly enforced.

The general absence of tramways, or street cars, in the center of London greatly simplifies the control of street traffic and renders it far easier and safer for a pedestrian to get about than it would be in a crowded American street. The chief danger to foot-passengers seems to be the motor vehicles, while a great annoyance, at least, is the omnipresent bicycles, many of which are motor-driven. There is a tendency for the motor vehicles and the bicycles to take what may be termed the neutral thread of the street for occasional spurts.

The excellence of the pavements of London is remarkable. So far as I have observed, wood predominates and it appears to be in greatest favor among cabmen. The wood blocks are quite commonly "sanded"; the material used ranging from granite, or other crushed stone screenings, to coarse sand and fine gravel. Sand or gravel bins of steel or iron-plates are built on the sidewalks close to the curbs, or on the isles of safety, and the sand is either thrown onto the street directly from there, or else from barrows.

The only time I have seen the sanding operation the sand was spread by means of long-handled shovels, the man giving the shovel a sweep through an arc of a circle and at the same time a dextrous turn of the wrist. In this way a small amount of sand is made to cover a large area. Much of the sand and gravel, naturally, is beaten into the wood by the heavy traffic, giving the tarred wood blocks the appearance, at a casual glance, of a hard tar-concrete.

Fortunately there has been but little rain since I reached London, but when rainy, it has seemed to me that the wood block pavements were dirtier and stickier than the asphalt, but not so slippery. Most of the asphalt which I have seen, however, has been on residential or other streets of comparatively light traffic.

Outside the central area of London I have seen a large amount of macadam, mostly in far better repair than macadam in America. Moreover, macadam is kept quite clean here. In fact, considering the enormous number of horses in London, the streets are kept wonderfully clean. In the busiest sections this is effected by the orderly system; boys or young men with a short-handled broom or brush and a short-handled shovel, or with a long-handled scraper, pick up the horse droppings almost as soon as they fall, and deposit them, for the most part, in orderly bins on the curb. These bins are generally of metal, small in ground plan and 5 or 6 ft. high. Boys or active men are essential to dodge the traffic.

Perhaps no greater contrast could be found than the 600 public water closets and washrooms scattered all over London, as compared with the 20 or 30, if so many outside of parks, in the whole United States. So far as I am aware, the only American cities with these public conveniences stated in order of the number provided, are Boston, New York and San Francisco; and in the last-named city the one station is due to the enterprise and generosity of the Merchants' Association. In the busiest parts of London the public convenience stations are sometimes not more than 500 ft. apart. Observations thus far made in the smaller towns around London show that they also are equally well supplied.

These stations are underground, generally beneath the street surface, and often entered from an isle of safety. As a rule, they have side walls lined with white enameled brick and urinals of white glazed earthenware, in the form of stalls. The floors are tiled or otherwise provided with waterproof pavements. The urinals are free. The water closets are available only on payment of a penny, or 2 cts., and are generally unlocked by dropping a penny in a slot. The lavatories or washrooms can be used on payment of 2d. to the attendant, which entitles you to a clean towel in a paper band wrapper, hot and cold water, hair brush and comb, clothes brush and hat brush. The attendant inquires whether you wish hot water, and at your request gives you an ample but still a limited quantity.

The limit on hot water just mentioned is an illustration of British economy as compared with American lavishness. In America we pay a maximum price in order that we may be hampered by no "petty restrictions": A 5-ct. fare for a ride of indefinite length on a street railway, as contrasted with 1-ct. or 2-ct. stages here, according to distance.

Since I reached London I have attended meetings of the Institution of Civil Engineers, the Royal Statistical Society, the Association of Sewage Works Managers and the annual dinner of Engineers in Charge; that is, in charge of the steam, electrical or similar plant of public institutions. All these meetings were interesting. I was particularly impressed by the meeting of the Institution of Civil Engineers. The papers read described grouting an old masonry barrage on the Nile and constructing a new barrage at Assiout, on the Nile. I estimated that between 200 and 300 were present at this meeting, and my surprise at the large number was exceeded only by my astonishment on seeing so many middle-aged and elderly men. At most of the ordinary meetings of the American Society of Civil Engineers which I have attended the majority of those present were juniors, or young men in, or just advanced out of, the associate class of members.

While as yet I have been outside of London but little, I have talked with a number of engineers and others familar with the whole country, including several officials of the Local Government Board, engineers and chemists in private practice, contractors, editors and political economists. All strongly confirm our common belief in America that municipal government is almost wholly free from bribery, stealing and direct corruption; but no one is at all satisfied with it, nevertheless, and aside from out-and-out dishonesty and jobbery, much the same general complaints are heard here as at home regarding the way municipal affairs are conducted. On the whole, however, I think we have been rightly informed as to the superior quality of British municipal administration; but for the most part we have heard only the best side of the story.

I have seen several refuse destructors, or furnaces, and a few sewage purification plants. My first impression is that we have more to learn from the refuse destructors than from the sewage works, and this impression is strengthened by my interviews with a number of the men here most prominently identified with sewage treatment. These men express such a diversity of views as to the methods of sewage treatment now in vogue and are bringing forward so many modifications of old methods that one cannot but feel that sewage treatment here is not on a very definite scientific basis. Only yesterday I saw the installation of a small system of detritus or septic tanks, small storage channel and triple contact filter beds in the morning, and in the afternoon plans for small works now under construction including a hydrolytic or new type of septic tank, to be followed by four-effect upward anaërobic filters. The effluent from this combined plant will go to an old installation of double contact or bacteria beds for aërobic treatment. A complete installation on this new scheme would have added to the hydrolytic tank and anaërobic filters a series of thin aërobic filter beds, placed one above the other. I have been promised details of this system for later use. Still another scheme is a single filter bed, constructed of thin slate resting at each end on blocks.

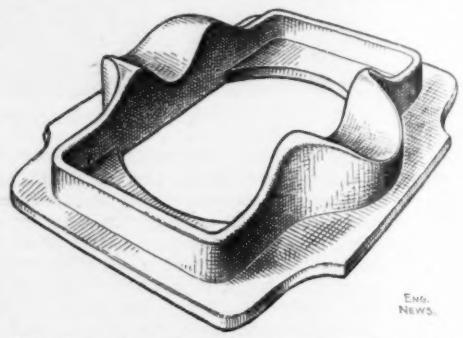
The material going to the refuse destructor I have visited, as well as all the refuse I have seen here in the so-called dust vans, contains little garbage proper as compared with the material arriving at an American garbage furnace.

In concluding this letter, it is a pleasure to acknowledge the universal courtesy and the general cordiality with which I have been received by engineers and others. The ice of formality may be a bit thicker here than at home, but, once it is broken, a well-accredited visitor, and particularly an American, is sure of a hearty welcome.

London, March 25, 1904. M. N. B.

A SUPPORT FOR WATER AND GAS SERVICE COCKS AND BOXES.

A simple device for supporting both corporation or curb cocks for private water or gas services and also the protecting service boxes is shown by the accompanying illustration. The support is of cast iron, weighs 7 lbs., and needs no further description than that afforded by the sketches. It is designed to hold the box in position and to keep it from becoming filled with earth at the base. The device has been patented and is manufactured by Mr. John R. Flinn, of Johnstown, Pa.



Support for Water and Gas Service Cocks and Boxes.

ANNUAL MEETING OF THE AMERICAN ELECTROCHEMICAL SOCIETY.

The St. Louis fair, which opens next month, seems to be casting its shadow before. At any rate, the fact that the attendance at the fifth meeting of the American Electrochemical Society, held last week at Washington, D. C., was numerically smaller than at the society's previous meetings, may be charged mainly to the exposition, for which, presumably, most of the members and guests are saving their energies. The total of members and guests in attendance was only about 90. And as the attendance, so was the scope and quality of the professional proceedings of the meeting also affected. However, the number of the papers, some fifteen in all, was quite sufficient and the discussion did not lag.

The meeting occupied three days, April 7 to 9, during which time four sessions were held. The lecture room of the Columbian University, on 15th and H Sts., was tendered for the sessions of the society, and here the meeting was opened on the morning of April 7 with a welcoming address by President C. W. Needham of the University. For the afternoons and evenings of the three meeting days abundant provision had been made by the local committee. At noon of the first day the

society proceeded in a body to the White House and there shook hands with President Roosevelt. In the afternoon, a special "Seeing Washington" car took the society all over the city, while a megaphone called attention to the points of interest, from a prehistoric negro shanty to the Pension Office, "locally known as Meigs' barn." That same evening, after the presidential address, there was a generally-enjoyed smoker. On the afternoon of the second day the party had the privilege of witnessing the shrinking of a jacket-tube on a 6-in. rifle, at the Washington Navy Yard. The gun shop, the cartridge shop, and the model basin at the yard were inspected, and later the National Bureau of Standards, at present housed in the building of the Coast and Geodetic Survey. A banquet at the Shoreham Hotel occupied the entire evening; Mr. H. W. Wiley, of the Department of Agriculture, acted as toastmaster with great success, and Prof. J. W. Richards, Col. S. Reber, Prof. H. S. Carhart, President Needham of Columbian University, and the retiring secretary, Mr. C. J. Reed, spoke in response to toasts. On the last day, after adjournment, most of the members visited Mount Vernon.

As appears from the reports submitted to the meeting, the society is in healthy condition. A large influx of new members brings the total of membership to about 600 at present, and correspondingly the society's assets have grown nearly to the $5,000 mark. The directors have arranged for an interchange of Proceedings with the English sister society, the Faraday Society. The Frenzel prize of $250 for work on the extraction of rare metals is still in the society's hands, not having been applied for up to the present.

A constitutional amendment, providing that retiring officers shall not be eligible for re-election to the same office had been before the society for some time past, and it was unanimously adopted by the meeting. While this amendment did not apply to the current election, yet the results of this election virtually carried the provision into effect, there being a change in most of the offices. Prof. Henry S. Carhart (University of Michigan, Ann Arbor), was elected President; the new Vice-Presidents are: Mr. C. J. Reed (Philadelphia, Pa.), Mr. E. G. Acheson (Niagara Falls, N. Y.), and Prof. C. F. Burgess (University of Wisconsin, Madison); Mr. S. S. Sadtler (Philadelphia, Pa.) was elected Secretary, and Mr. P. G. Salom (Philadelphia, Pa.) was re-elected Treasurer. The newly-elected managers are Messrs. W. H. Walker (Boston, Mass.), C. E. Acker (Niagara Falls, N. Y.), and E. Weston (Waverly Park, N. J.).

PRESIDENTIAL ADDRESS.

Retiring President, J. W. Richards (Lehigh University, So. Bethlehem, Pa.), delivered an address on "Electrochemical Advance" at the second session of the meeting. He treated his subject in three parts: experimental research, development of theory, and development of industrial application. The first, experimental research, has been rather haphazard and disjointed in the past, due partly to independent working and lack of exchange of ideas. In consequence, large gaps exist in our experimental knowledge of electrochemistry, which it is the work of the immediate future to fill. The theory of the science is in no more satisfactory state. Theoretical beliefs are in a state of transition at present. President Richards' own view is that "it is a mistake to hold the ionization theory to be true," and his leaning is toward the belief that "a dissolved substance is in an abnormal physical state rather than an abnormal state chemically." Turning to industrial applications of electrochemistry, he reviewed the recent achievements in this field, including as most recent the production of silicon —a material which makes up one-fourth of the earth's crust. Of future developments, most important would be a mastery of the metallurgy of the strong metals. Here the prospects are not highly promising; the commercial reduction of iron ore to pig iron is very far from a possibility. But for making certain grades of steel electric furnace working is already a commercial success, and this field may possibly be extended in the future to cover other qualities of steel. There is promise, also, in the use of electricity for supplying the last increment of heat in many metallurgical operations, the part that is most costly with present methods. In the field of materials now wholly unavailable to man, we may expect that electric furnace processes will give us calcium, which forms so large a part of the solid earth, and later, perhaps, beryllium, a substance by no means rare and promising valuable qualities.

PROFESSIONAL PROCEEDINGS.

The first two papers of the meeting dealt with a subject which is very important to electrical science as a whole—the provision of a standard primary cell, that is, a source of electric potential which might be used the world over as a standard by which to measure voltages. The standard unit of electric potential (of which one hundred million make one volt) is defined as the potential generated in a conductor which cuts one magnetic line of force per second. This definition does not offer a practical means of measuring potentials. A better means is found in Ohm's law, connecting potential, current-strength, and resistance; if two of these quantities can be accurately measured, the third is determined. The ohm, the unit of resistance, is defined in such a way as to be very accurately reproducible. The unit of current, the ampere, is conveniently defined by the amount of silver deposited from a silver solution by a given current in a given time. This method of measuring current involves some small uncertainties, for example in the amount of irregular action taking place at the electrodes; yet it is accurate enough to allow of repeating an observation with a variation no greater than 1 in 40,000, which is more accurate than our knowledge of the value for the rate of silver deposition. However, the method has the defects of inconvenience and of narrow range, both in the amount of current and in time. A precise standard source of electric potential is therefore highly necessary to electrical industry. The so-called Clark primary cell has hitherto been accepted as such a standard, and its voltage has been quite caretully determined. It possesses certain undesirable qualities, principal among them the large temperature coefficient, or change of voltage with change of temperature. The Weston or cadmium cell has been found to be far better, in this respect, and at the International Electrical Congress, to be held at St. Louis in September of the present year, its adoption as standard cell will come up for discussion.

Mr. F. A. Wolff (Washington, D. C.), of the National Bureau of Standards, presented a paper on "Standard Cells," in which he described the work done by him for the Bureau in investigating the sources of error in the cadmium cell. He found that all the constituent elements of the cadmium cell may be readily obtained in satisfactory purity, except mercurous sulphate. For producing this material, he finally devised an electrolytic process which consists in passing an electric current between electrodes of mercury in sulphuric acid. It is necessary to keep the surface of the mercury exposed to the acid by stirring. The mercurous sulphate obtained is highly satisfactory.

It may be noted that the British National Physical Laboratory has, during the past year, also been at work to find a satisfactory method for preparing mercurous sulphate.

Exactly similar work was reported on by Prof. H. S. Carhart and Mr. G. A. Hulett (Ann Arbor, Mich.) in a paper entitled "Preparation of Materials for the Cadmium Standard Cell and Its Construction." The work of Professor Carhart on the cadmium cell has been remarkably precise and thorough. Much of the work he had previously reported upon, including the construction and use of a very accurate dynamometer for measuring directly the potential of the cell. The present paper describes the best methods for preparing the various materials of the cell, discusses the influence of variations in the mercurous sulphate, and describes the method finally devised to prepare it in great purity, this being the same electrolytic method as devised by Mr. Wolff. The subject of most practical form of construction of the cell was studied, resulting in the adoption of the H form of cell, both vertical legs being sealed at the top after the materials are introduced. Several cells of this kind were then prepared, at different times and from batches of material prepared by different persons. Tables and curves of the resulting voltages were given, showing virtually absolute agreement when properly prepared materials were used. Cells made with electrolytic mercurous sulphate were found to require no aging, such as the other cells required to drop to their constant voltage. The perfected cell has a constant voltage of 1.01908 international volts when measured at 21.1° C. It should be noted that its temperature coefficient is about one-thirtieth of that of the Clark cell.

In discussion Mr. H. N. Potter (New York, N. Y.), asking whether the kind of glass used for the cell has any influence, Prof. Carhart replied that this factor had been kept fairly constant in the work, but that he would not expect much variation due to the glass. He also urged that this form of cell, with saturated solution, be called the cadmium cell, while the unsaturated cell (saturated at 4° C., and having zero temperature coefficient) be called the Weston cell. The former is to be recommended for adoption by the International Electrical Congress as the primary standard of potential, while the Weston cell will probably be better as a secondary or working standard.

A paper entitled "Single Potentials of the Halogen Elements," Mr. W. R. Mott (Madison, Wis.), was read without discussion. The paper gives calculated single potentials of various metals, found by substracting the single potential of the halogen from the decomposition voltage of the salt in dilute aqueous solution.

Another theoretical paper, "The Composition and Resolution of Voltages," by Prof. J. W. Richards (South Bethlehem, Pa.), discussed the action in electrolytic cells where several actions proceed simultaneously. In such a cell, each independent action corresponds in general to a different potential, acting in one direction or the other, and differing from the resulting voltage across the cell. The author's view is that each action contributes a potential equal to the independent voltage of that action multiplied by that fraction of the total current of the cell which is absorbed in that action. As an example, a case was taken from electrolytic copper refining, where the anodes contain various metals which go into solution. If an anode containing iron, nickel and zinc, combined with the copper, dissolves in the ratio: 90 Cu, 5 Fe, 1 Ni, and 1 Zn, then a calculation on the above principle shows that the impurities give an assisting voltage of .065 volts. The author suggested that the problem should be considered from the energy standpoint: each independent action (in the above case the solution of copper, iron, nickel, and zinc from the anode) absorbs energy in proportion to the relative quantity of that action and to the energy absorbed when that action proceeds independentiy; the total energy required will then be the sum of these partial actions, and divided by the total current it will give the resultant voltage. Thus, in depositing copper from a solution, the voltage is constant until the solution becomes impoverished so that hydrogen is liberated, when the voltage gradually rises until it again becomes constant at the hydrogen value when the copper is entirely exhausted. That is to say, an electrolytic action, when combined with another, may take place at lower or higher voltage than is required for that action proceeding independently.

This paper was discussed by Messrs. L. Addicks (Perth Amboy, N. J.), W. R. Mott (Madison, Wis.), C. Hering (Philadelphia, Pa.), C. J. Reed (Philadelphia, Pa.), R. Gahl (Philadelphia, Pa.) and others. Mr. Gahl disagreed with the principle formulated by the author, and believed that if the resulting voltage differs from the highest voltage required for any of the actions independently, this was chargeable to alloying or other vitiating influences. Local action at the electrodes, solvent action of the electrolyte, etc, were cited by others as affecting the problem. The author, however, stated that alloying, local action, etc., had been expressly neglected in his paper, and for complete consideration these would have to be taken into account. Several cited observations which tended to support the author's argument.

An iconoclastic paper was read by Mr. C. J. Reed (Philadelphia, Pa.) under the title "Molecular Conductivity," in which he attacked one of the basal arguments of the dissociation theory. In part, this theory rests on the observation that if a solution of a given salt be successively diluted, its specific electric conductivity, referred to equal weights of dissolved substance, increases toward a limiting value for infinite dilution. From this fact (taken together with many other facts) it is reasoned that only in very dilute solutions is all of the substance dissociated into ions; moreover, the degree of ionization is calculated from the observed conductivities, referred to the conductivity of the infinitely dilute solution. Mr. Reed objects that the procedure is illogical because it does not refer to a body of standard length and cross-section; for, conductivity must be compared on the basis of equal bodies, say one cubic centimeter, whereas in the above measurements the body measured increases in size as the dilution increases (the length alone being maintained constant). He holds that not only must the conductivity be referred to the same size of body, but also the influence of the solvent must be considered. Thus, pure water and pure hydrochloric acid have no appreciable conductivity, but mixtures of them do conduct electric current; it must be wrong to reason that the increased conductivity of water when a little H Cl is added is due wholly to the acid. The author declared that all authorities on physical chemistry share in the error which he points out.

Mr. E. F. Roeber (New York, N. Y.) came to the defense of the ionic theory with a mathematical development intended to show that no such error was committed, or at least that the procedure objected to gives results which agree with those calculated from the lowering of the freezing point, the raising of the boiling point, etc. Mr. R. Gahl stated that the conductivity of the solvent (water) is properly taken account of in the ionization calculations, but that except for extremely dilute solutions it is negligible. Prof. J. W. Richards, however, agreed with the author to the extent of considering it objectionable to assume that the conductivities of solvent and solute may be combined by simple addition or subtraction.

It will be remembered (see Eng. News, Sept. 24, 1903) that at the last meeting of this society Prof. W. D. Bancroft presented the results of an experimental investigation of the principal factors affecting the cost of electrolytic copper refining.

The investigation was summed up in three recommendations: Use a current density of 31.5 amperes per sq. ft.; use covered tanks; keep the temperature of the solution at 70° C. It appeared that the practice of the largest refineries in the country did not accept these recommendations, and in the discussion at that meeting two representatives of refineries, Mr. L. Addicks and Mr. B. Magnus, advanced arguments to show that the problem was more complex than Prof. Bancroft's analysis assumed.

At the present meeting Mr. L. Addicks (Perth Amboy, N. J.) gave some further discussion of the problem, under the title "The Economic Balance in Electrolytic Copper Refining." He gave no calculations but confined himself to enumerating the factors which affect the cost of refining, and pointed out their interrelation. He considers the arbitrary variables of the problem to be: Tank resistance, Age of electrodes, and Current Density. The last-named variable is treated thus:

> From the standpoint of current density the problem becomes largely one of output. With any given plant the simplest way to increase the output is to raise the current density. Even our largest plants still have a respectable proportion of fixed charges, and the distribution of these over a larger tonnage is always an attractive proposition. If interest charges are involved, we gain here as well. With a rising density, however, we are met by a rapidly increasing power cost and a roughening of the deposit. The small counter e. m. f. present keeps the power needed per tank from increasing quite as the square of the current density, but it approaches this ratio. The rougher cathodes require a readjustment of several of the conditions previously outlined, involving a corresponding increase in costs in order to maintain the current efficiency. The circulation must be pushed, and this is followed by increased silver losses in the cathode—a charge which mounts rapidly. One-tenth ounce or 1.5 ounces per ton might be equally good work for different sets of conditions. The increased energy expended in the cell brings the temperature up and helps the tank resistance. As the density increases the relative cost of operating insoluble anode tanks decreases.

The author's conclusion is that the problem of economical balance of conditions must be solved by actual trial for each case separately. A brief discussion (by letter) from Prof Bancroft and a reply by Mr. Addicks brought no further clearness.

A paper by G. M. Westman (New York, N. Y.) entitled "Remarks on Prof. Richards' Theory of Heat of Neutralization" was ultra-theoretical and controversial in character. It opens with the passage:

> When a layer of water is poured over a solution of ordinary salt, the salt itself begins to move upward in the solution until the whole mass is uniform. This phenomenon, which is called diffusion, depends on the centrifugal force of the rotary movements of the molecules, which force acts against gravitation.

The author builds up his objections to Richards' theory from this and similar statements, which he calls "facts." Prof. H. S. Carhart entered protest against calling such statements facts, and characterized them as "the most extreme theory," using "theory" apparently in the sense of "speculation."

A paper by Mr. L. A. Parsons (Salt Lake City, Utah) on "The Energy of Ions," was also purely theoretical and controversial, the author taking issue with some objections advanced by Mr. C. J. Reed against parts of the ionic theory. The paper was briefly discussed by Messrs. E. F. Roeber, R. Gahl, G. L. Cabot, J. W. Richards and C. J. Reed, of whom the two first-named upheld the author's views; nothing of moment was brought out, however.

THE ALUMINUM ELECTROLYTIC CONDENSER.

A paper on this subject by Mr. C. I. Zimmermann (Madison, Wis.) presented a considerable amount of interesting information.

It is well known that aluminum when used as electrode in an electrolytic cell acquires the peculiar property of allowing current to flow freely from aluminum to the liquid, but opposing high resistance to passage of current from liquid to plate. This property is utilized for constructing electrolytic rectifiers, which transform an alternating current into direct current, suppressing one-half of the alternating wave. By arranging four such cells in a peculiar way, the so-called Graetz arrangement, both halves of the alternating wave are utilized and changed into direct current. It has also been known for some years that if two aluminum electrodes be put into opposition, the arrangement acts as a condenser for alternating current, preventing the passage of current but generating a leading condenser current. The author has very fully investigated the properties and possibilities of such condensers, and believes that they are of sufficient value to give the device a commercial future in competition with the tinfoil condenser.

The peculiar property of the aluminum plate, already referred to, has been known to be due to an insulating film, probably an oxide, which forms on the surface. The breaking-down voltage of this film varies with the nature of the electrolyte and other factors. Usually it is only comparatively few volts, but the author has succeeded in obtaining a cell in which the breaking-down voltage was 1,250 volts. In general, the voltage limit on a single cell will be about 150 volts, if good efficiency as a condenser is a consideration. The thickness of the film also varies greatly, from about .000005 cm. to .00005 cm. Its dielectric constant, or specific inductive capacity, is probably about 10 times that of air. The aluminum electrolytic condenser may be operated commercially at efficiencies near 95%, though the author has in some cases obtained higher efficiencies. The capacity is about ¼ to ⅓ microfarad per square inch of surface of one electrode. In an alternating circuit the maximum electric pressure on the surface film of the aluminum will be the full maximum voltage of the circuit. The cell charges and discharges only one-half, retaining a constant charge which remains in the condenser on disconnecting it from the circuit. This charge resides in the plates, the electrolyte being neutral. The energy losses in the cell were analyzed by the author, who classified them into resistance loss, leakage loss and film loss. The total losses increase about proportional to the frequency of the current. The electrolytic condenser has the advantage that punctures automatically repair themselves.

Mr. W. R. Mott (Madison, Wis.) in discussing the paper, gave the above-cited figure of 10 for the dielectric constant. He named the factors influencing the thickness of the film, i. e., voltage, electrolyte and (probably) past history of the aluminum plate. The thickness of the film was determined by interference colors; no quantity sufficient for analysis has yet been obtained. The leakage loss in the aluminum condenser is due to conduction by way of minute punctures of the film, which constantly occur; the loss remains small and constant, however, at increasing voltage, until the break-down voltage is reached. At this point minute sparking may be detected under the microscope, and the loss rises rapidly.

Referring to the Graetz rectifier arrangement, he stated that it was invented and patented in America in 1891 by Mr. T. D. Bottome (Pat. Sept 1, 1891).

Mr. W. S. Weedon (Schenectady, N. Y.) presented an interesting paper, "A Contribution to the Study of the Electric Arc," dealing with metallic arcs. Arcs between metal electrodes are nearly always so-called flame arcs, as contrasted with the crater arc of the ordinary carbon arc light. One of the points investigated by the author is whether the transfer of material across the arc corresponds to anything like the Faraday equivalents for transfer of material in electrolytic cells. He finds that this is clearly not the case. The transfer is usually very small, particularly with water-cooled electrodes where but little loss by vaporization occurs. The positive electrode usually gains in weight while the cathode loses, but sometimes the case is reversed. The work was conducted principally on arcs in gases other than air. The purely scientific nature of the work precludes our giving more extended notice of the paper.

"Phenomena Observed in the Use of a Copper Voltameter," by Messrs. Barry MacNutt (Bethlehem, Pa.), and Isaac Adams (Cambridge, Mass.) was read by Mr. MacNutt. The paper described difficulties which were encountered in making some precise measurements of current by means of a copper voltameter. It was found that when the cell was left standing for a short while, the polarization increased instead of decreasing, as it does in other electrolytic cells. The dissolving action of the electrolyte on the deposit introduced another error. The work with this voltameter was carried out to the fourth decimal place, but Mr. C. Hering (Philadelphia) objected that this was futile, since the electrochemical equivalent of copper is not known as accurately as the fourth decimal place. Moreover, the dissolving action upon the deposited copper is involved in so many factors that it cannot be accurately allowed for.

THE ELECTROLYTIC PRODUCTION OF PURE IRON.

Interesting work was reported by Prof. C. F. Burgess and Mr. C. Hambuechen (Madison, Wis.) in a paper entitled "Electrolytic Iron." They have investigated very thoroughly the practicability of producing pure iron by electrolytic refining, and find, in brief, that it is not only a laboratory possibility, but a commercial one, with regard to both quantity and cost, provided a sufficient demand for the product should arise. At present the only commercial use for electrolytic iron is in "steel facing" of dies and electrotypes.

The process is simple electrolysis of an iron solution from iron anodes. The best solution to use is ferrous sulphate with a small admixture of ammonium sulphate. The current density should be about 6 to 10 amperes per square foot of cathode surface, and slightly lower at the anode. The temperature of the bath should be about 30° C. The potential across a cell will be slightly less than 1 volt. Cast steel or wrought iron of commercial quality may be used as anodes. The cathode starting sheets used were thin plates of sheet iron, carefully cleaned of rust and scale. In

the beginning of the experimental work, "treeing" gave trouble, producing irregular deposition and short-circuiting the cell. This has been overcome, however, so that now the authors have run four weeks with one cathode; at the end of this period the cathode is about ¾-in. thick, and its surface is quite rough and nodular, so that the deposition has reached its limit of practicability. The efficiency of the electrolysis is near 100%. About 2.2 lbs. of iron is produced per kilowatt-hour. The iron which is deposited adheres loosely to the starting sheet, and is readily removed.

During the past six months the authors have had in continuous operation three tanks, each 8 x 13 ins. and 15 ins. deep, each containing two anodes and one cathode. They have produced a total of about one-half ton of electrolytic iron.

The purity of the iron produced is about 99.9%. A large amount of hydrogen is occluded in the metal, and this makes it very brittle and so hard that it can be filed or sawed only with great difficulty. The hydrogen is readily driven off by heating to white heat. The melting point of the pure iron produced is probably near that of platinum, but it is very difficult to determine because the iron absorbs impurities very easily at high temperatures. Thus, the iron is easily melted in a graphite crucible, due to the absorption of carbon, but when the melting is tried in a silica crucible the latter melts before the iron. The magnetic properties of the iron, i. e., permeability, hysteresis, etc., are greatly dependent upon the amount of hydrogen occluded, and the results so far obtained in this matter are therefore variable.

The authors believe that the cost of this refining should not exceed ½-ct. per lb., and that the product should be able to compete with the purer kinds of iron selling above 3 cts. per lb.

In discussion, Mr. C. Hering (Philadelphia, Pa.) said that electrolytic reduction of iron directly from the ore is commercially impracticable, partly because the fixed plant (tanks, etc.) would be too extensive, partly because there is no material which could absorb the liberated oxygen and return this energy to the cell. The refining of iron is therefore the only way in which electrolytic processes can be applied to iron metallurgy. He asked why cast iron could not be used for anodes in this process; a year ago he was told by Professor Burgess that cast iron gives trouble in operation. He believed the process would prove very much more valuable if cast iron could be used. Mr. W. M. Johnson (Iola, Kan.) attributed the failure of cast iron to the large percentage of impurities in this metal, which would not only cause the anodes to slime too rapidly, but would also result in their falling to pieces and thus producing an excessive amount of scrap. Speaking on the subject of proper solution, he stated that he had made experiments on electrodeposition of iron some time ago, and had found that the best solution was ferrous sulphate with ammonium salts and a small addition of some organic acid of small degree of dissociation.

Mr. Johnson then presented a note on "Reaction Temperatures in the Reduction of Zinc Oxide." The note described a small graphite crucible with inserted thermo-couple, which was devised to measure the reaction temperature, and gave the temperatures observed. The note resulted in some discussion of whether or not carbon is slightly vaporized at high temperatures (about 1,000° C. in this case). Mr. Johnson believed carbon vapor plays a part in starting the reaction in the reduction of zinc oxide, and referred to the blackening of incandescent lamp bulbs as showing that carbon vaporizes slightly. However, Mr. C. J. Reed, among others, held that carbon does not vaporize.

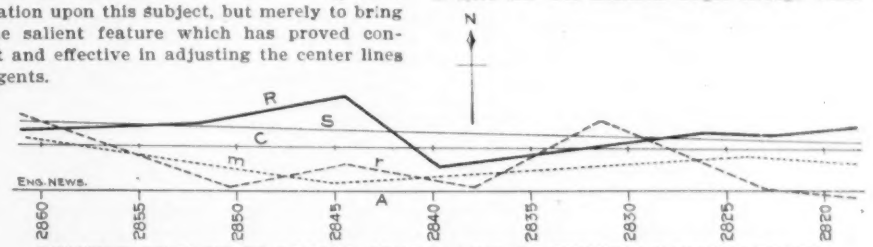
In addition to the preceding, the professional program comprised several papers which were read by title only. Among these are "Electric Smelting Experiments for the Manufacture of Ferro-Nickel from Pyrrhotite," by Mr. E. A. Sjöstedt (Sault Ste. Marie, Ont.); "Observations on the Preparation of Electrolytic White Lead," by Mr. C. F. Carrier, Jr.; "Dissociation by Means of the Alternating Electric Current," by Mr. W. H. Davis (Boulder, Colo.); and "A Decimal Index for Electrochemical Interests," by Mr. A. L. Voege (Zürich, Switzerland).

PRACTICE IN RE-ESTABLISHING AND MONUMENTING A RAILWAY CENTER LINE.

By Albert I. Frye,* M. Am. Soc. C. E.

The value of re-establishing and monumenting railroad center lines is recognized by the legal, real estate and engineering departments of Eastern railroads as an economic necessity. It crystallizes and harmonizes the actions of local surveyors in their property surveys adjacent thereto and is a fitting preliminary to side-monumenting, adjustment of track and various operations incident to new work.

The writer does not wish to enter into a general dissertation upon this subject, but merely to bring out one salient feature which has proved convenient and effective in adjusting the center lines of tangents.



EXAMPLE OF PLOT TO LOCATE CENTER LINE OF EXISTING RAILWAY TRACK.

An "instrument line" is run, say, 7 ft. north of and parallel with an "assumed center line" of right of way, and all monumental land marks, such as old monuments, fences, track centers, conters of structures, buildings, *etc.*, which may be *reasonably supposed* to have been placed at *certain* distances from the old center line, are located. The *ranges* and *stations* (rectangular coordinates) of these points are calculated with reference to the "assumed center line."

On a longitudinal scale of, say, 800 feet to an inch the "assumed center line" is laid off, the lateral scale being 1 ft. to 1 in. The various points which have been located and calculated are then plotted from the "assumed center line" the lateral distance being the difference between the range as calculated and its *supposed* original distance from the old center line. For instance, if the range of an old fence were found to be 49.75 ft. north of assumed center line whereas it was supposed to have been placed 50 ft. distant, then the fence at that point would tend to establish the center line 0.25 ft. south of the "assumed" and the point would be platted "range 0.25 south."

It is essential to classify the various landmarks used and to further distinguish those found on the north side of right of way from those on the south, *connecting* each class on either side of right of way by its characteristic (colored) line.

The following notation has been used with success:

A.	(black line)	=	Assumed center line.
R.	(full brown line)	=	Railroad fences on north side of right of way.
r.	(dotted brown line)	=	Railroad fences on south side of right of way.
P.	(full yellow line)	=	Private fences on north side of right of way.
p.	(dotted yellow line)	=	Private fences on south side of right of way.
M.	(full blue line)	=	Monuments on north side of right of way.
m.	(dotted blue line)	=	Monuments on south side of right of way.
S.	(full green line)	=	Existing structures.
T.	(full pencil line)	=	Existing tracks.
C.	(full red line)	=	Adjusted center line.

Center line as would be established by

The accompanying simple sketch will serve to illustrate.

The adjusted center line is best perfected (after all the others are platted) by the use of a fine thread and in a manner similar to establishing grades, on profiles. The lateral scale adopted—1 in.=1 ft.—insures great accuracy. It must be remembered that all the lines shown are actual possible center lines crossing and recrossing each other in a maze of net-work and are in no sense offset lines, hence the final adjustment is a direct one.

The object in distinguishing the north monumental points from those on the opposite side of right of way is to avoid encroachment.

As frequently happens with long "tangents" run in the early days the readjustment necessitates a slight "bend" the position of which should be fixed at some "round" station by a stone monument. The semi-graphical method above described readily points to the position of such a bend, and leaves a permanent record of existing facts and conclusions drawn. The center line being thus fixed is permanently monumented on the ground.

*Room 723 Central Bldg., New York City.

THE MASTER CAR BUILDERS' DROP TESTING MAchine, which for some time has been in process of erection, is now completed and ready for use. By direction of the Association, the machine has been installed at the laboratory of Purdue University, Lafayette, Ind., where it will be operated under conditions similar to those prevailing in connection with the Master Car Builders' brake-shoe testing machine and the Master Car Builders' air-brake testing rack. The machine may be used not only by Committees of the Master Car Builders' Association, but also by individual railway companies and manufacturers as well.

The drop weighs 1,650 lbs., and is so designed that supplemental pieces may be added to increase its weight to 2,000 lbs. The maximum height through which the drop may fall is 50 ft. The machine may be used in testing materials of any kind, provided that the form of the part is such as will permit it to be located under the drop. It is especially designed to test couplers, draft gears, axles, rails and bolsters.

Parties desiring to use the machine should communicate with Prof. W. F. M. Goss, Dean of the Schools of Engineering, Purdue University, Lafayette, Ind.

THE MOST SERIOUS RAILWAY ACCIDENT of the week was a rear collision on the Chicago & Northwestern Ry. near Maywood, Ill., on April 7. Three passengers of the forward train were killed outright, and several others fatally injured. The forward train, a through passenger train from the West, was running late, and at the time of the accident had been stopped by block signal. A dense fog prevailed and a fast mail train following ran into the passenger train, crushing the two rear cars.

A REMARKABLE ACCIDENT occurred at the new dam at Boonton, N. J., of the East Jersey Water Co., on April 11. The dam is nearly completed and the reservoir is partly filled. One of the valves in the 4-ft. outlet tunnel through the dam became jammed some time ago, and in order to get at the valve for repair it was decided to try to close the inlet end of the pipe with a large wooden ball, properly weighted to sink. After several attempts, the ball was caught by the current and carried against the pipe-opening, as intended, but it caught an end of the rope used in lowering it and jammed the rope against the pipe, so that the opening was not quite closed. A diver sent down reported that the leakage might be stopped with sandbags, though the work would be very dangerous. He undertook to place the bags, and had the work nearly completed when his foot was caught by the current and jammed in among the sand bags. Since then it has been impossible to release him, in spite of every effort, and at this writing he has been in the water some 48 hours. There is little or no chance of his being rescued alive, although the air supply has been kept up constantly.

A MONUMENT TO GRAMME, the electrician, is proposed by the Association of Graduate Engineers from the University of Liege (Belgium) and the International Street Railway Union. The monument is to be erected in Liege, where he was born, and subscriptions for this purpose are now being invited.

IRRIGATION IN NEW MEXICO in 1902, according to the U. S. Census Bureau, was applied to 9,285 farms, and the total irrigated area was 254,945 acres. There were 1,246 different irrigation systems with 2,846 miles of main canals and ditches, and the total investment, including dams and all works, was $4,301,915. The average investment per acre irrigated was $16.87. At the present time most of the normal flow of the streams is already appropriated for irrigation and the extension of irrigation is therefore dependent on the storage of water on a large scale.

ENGINEERING NEWS

A Journal of Civil, Mechanical, Mining and Electrical Engineering.

Entered at the New York Post-Office as Second-Class Matter. Published every Thursday at St. Paul Building, 220 Broadway, New York, by

THE ENGINEERING NEWS PUBLISHING CO.

PUBLICATION OFFICE, 220 BROADWAY, NEW YORK.
CHICAGO OFFICE, 1636 MONADNOCK BLOCK.
BOSTON OFFICE, 170 SUMMER ST.

FOREIGN AGENTS.

C. MITCHELL & Co, Snow Hill, Holborn Viaduct, LONDON.
RUDOLF MOSSE, BERLIN and HAMBURG, GERMANY.
JOHN F. JONES, & CIE, 31 Rue de Faubourg Montmartre, PARIS.

SUBSCRIPTION RATES: United States, Canada and Mexico, One Year, $5.00; 6 months, $2.50; 10 weeks, $1.00. To all other countries in the Postal Union: Regular Edition, One Year, $8.00; Thin Paper Edition, One Year, $6.75. SINGLE COPIES of any number in current Volume, 15 cents.

In requesting change of mailing address, state BOTH old and new address; notice of change should reach us by Tuesday to be effective for the issue of the current week. The number on the address label of each paper indicates when subscription expires, the last figure indicating the year and the one or two preceding figures the week of that year; for instance, the number 325 means that subscription is paid to the 32d week (that is the issue of Aug. 10) of the year 1905; the change of these figures is the only receipt, unless one is sent by special request.

ADVERTISING RATES: 20 cents a line. "Want" and "For Sale" notices, special rates, see pages 26-27. New copy for standing advertisements should be received ten days in advance of publication; new advertisements, Monday morning. The pages containing "Want" advertisements are held open until Tuesday, and those with "Proposal" and "For Sale" advertisements until Wednesday at 11 a. m.

In connection with the movement toward better municipal government in this country, a considerable amount of matter has been published during the past dozen years concerning the conduct of municipal affairs in European cities. These articles, almost without exception, however, have treated the matter from a popular standpoint, and where they have dealt with matters of engineering they have done so under the limitations of the non-expert writer. Of the engineers in active practice in this country a few only have the opportunity of going abroad to study at first hand the practice of European cities in matters of municipal engineering, and of the results of such studies very little has been published.

In view of these facts, it has seemed that a systematic study of current practice in municipal engineering in European cities and the presentation of those features which American engineers may most profitably copy would be of much interest and value to the readers of Engineering News.

With that end in view, Mr. M. N. Baker, who has been for more than a dozen years in charge of matters of municipal engineering and sanitation on the editorial staff of this journal, was delegated to go abroad early in the year and spend several months in the investigation of European practice in municipal engineering, giving particular attention to work in Great Britain. The first of a series of letters reporting results of his observations appears in the present issue.

In Mr. Baker's absence it is, perhaps, permissible for us to say that he ranks among the foremost authorities in this country in the field of sanitary engineering. He is a keen observer and a thoroughly impartial and conservative student and critic of municipal engineering practice. The very many engineers who are personally acquainted with him will watch with especial interest, we are sure, his reports of the work of municipal engineers abroad.

The question of stronger construction for passenger car frames has been revived recently by a series of collisions in which the ordinary day cars were wrecked and the Pullman sleeping cars were practically uninjured. The remarkable immunity from accident which Pullman passengers enjoy has been frequently noted. Of course one reason why Pullman cars generally escape with so little damage in accidents is that they are usually at the rear of the train. Another reason is that in a collision the energy of the moving mass is expended in crushing the weakest parts, and the lighter-built day coaches cripple before the heavy Pullmans. The same rule applies as in the case of steel freight cars, which are generally saved in a wreck by the crushing of old wooden cars in the same train. If all the cars in a train were of steel, it may well be expected that a wreck would cause more damage to steel frames than at present.

But the fact that if all cars in a train were equally strong, the Pullman cars might not fare so well as at present is no argument against better construction for other classes of passenger rolling stock. The mail clerks have long demanded safer cars, and with good reason, as postal cars are usually at the head of the train, and although their underframing is usually heavier than that of ordinary passenger cars, the upper framing is very light, forming simply a frail box, with little reinforcement or bracing. Steel frame construction is frequently urged as an improvement in this line, and has much to recommend it, although it is not in itself sufficient to meet the necessities of the case. The principal points to be aimed at in improved car construction are: First, to have all cars of equal strength, and, second, to hold the cars of a train together, so that the whole train will take the shock of a collision as one mass. This latter point can be obtained to a large degree by the use of vestibuled ends to all cars in passenger trains, as noted below.

It is probable that the question of steel-framed passenger cars will before long be given consideration by the Master Car Builders' Association. Since steel underframes are so largely used for freight cars and heavy electric cars, their adoption for passenger cars would seem to be in the general line of improvement in car construction. In collisions, however, one of the most frequent and disastrous features of the accident is the telescoping of the cars. One car will rise high enough for its platform and underframe to clear that of the next car, and then drive back through the former, which has only its light side framing to resist the shearing force of the platform backed up by its sills, like a battering ram. A good deal has been done within recent years to reduce this effect of the collision by means of improved couplers and by the application of vestibule ends to ordinary passenger cars. With such construction it is almost impossible for one car to telescope another to a serious extent, but the majority of ordinary cars are still unequipped with this device, although for Pullman cars and such special cars as dining and parlor cars it is practically universally used. In very many cases, again, the tender crushes into the engine cab and the first car, the connection between engine and tender being generally very poorly designed to withstand collision strains. Some ten years ago when the vestibule idea was new, it was proposed to fit a dummy vestibule on the back of the tender, butting against the vestibule diaphragm of the head car. The Pullman train exhibited at the Columbian Exhibition in Chicago, in 1893, had this arrangement, but it has never come into general use. We know of but one railway now using it, and that is the Cincinnati Southern Ry., as noted in our issue of Feb. 20, 1902.

Coming back to the question of steel framing for passenger cars, we may consider the framing of the sides and roof. Interesting examples of steel framed cars are now in use in the Chicago suburban service of the Illinois Central R. R. (Engineering News, Sept 3 and Nov. 26, 1903), and are giving satisfactory service. With steel sills and continuous steel plates or beams over the windows, and with vestibuled ends to the cars, it would seem that the risk of telescoping might be much reduced. In the Illinois Central cars, there is also a system of bracing in the end panels of the sides, which thus serve as collision bulkheads. We recently discussed this matter of steel-framed passenger cars with Mr. A. W. Sullivan, Assistant Second Vice-President of the Illinois Central R. R., and print below his reply to a letter of specific inquiry on certain points. In regard to his remarks on the inside finish, however, we would note that in discussing the cars above referred to (in relation to the handling of suburban traffic), in our issue of Nov. 26, 1903, we suggested the use of such material as papier mache or artificial leather for the inside finish. With this remark we give Mr. Sullivan's letter, as of particular interest at this time:

Concerning the comparative cost of steel frame and wooden passenger cars, it is not possible as yet to give any definite information, as our first order for the steel frame cars is not yet completed. Our people are of the opinion, however, that there will not be very much difference between the cost of steel and wooden cars, although it is probable the steel construction will be somewhat more expensive than the wood.

The interior and exterior finish is alike for either car, and can be made as plain or as elaborate as may be desired, so that the cost of the finish is not an essential item where wood finish is used, and it is probable that wood finish will continue to be used for a long time. It is possible, however, that when the steel frame idea becomes better understood by the public, the wooden sheathing will be omitted from the outside of the cars, but it will be necessary to retain it for the interior as a protection against the variations of heat and cold; no other material known being so good for this purpose as wood.

As to the relative weight of steel and wooden cars, steel construction will be heavier than the small and lightly constructed wooden cars, which have been used for many years, but it will be lighter than the heavy wooden construction used in Pullman cars and in postal cars.

Unquestionably the best method of preventing the smashing of cars in train accidents is to construct the frames of steel, for the reason that while a steel-frame car may possibly be wrenched and distorted in an accident, it can never be knocked to splinters like a wooden car. The resistance to shock of both the upper frame and the under frame will be greater than that of a wooden car, and the passengers will therefore be afforded greater protection. The quantity of wood used in steel frame cars is very much reduced, and in the event of fire there is much less to burn. The use of wood in such cars being confined to the interior lining and exterior sheathing, all of which is thin stuff, it can readily be made fireproof and probably entirely incombustible. Steel frame cars such as we are constructing, with floors of ¼-in. steel plate, contain no wood whatever in the underframe, which is a great safeguard from fire in case of train accidents, as in nearly all cases, now that car stoves have been done away with, fires in wrecks start externally and usually from underneath the car.

The paper by Mr. Gunvald Aus, entitled "Some Comments on Building Laws," which we print in this issue, makes suggestions regarding the provision of wind bracing and erection bracing in high columns and the restriction of the use of cast-iron columns which appear to us to deserve hearty commendation. We are compelled to strongly dissent, however, from Mr. Aus's proposal to reduce the live loading of floors in buildings intended for residence purposes below the present minimum of 60 lbs. per sq. ft. It is quite true, as Mr. Aus says, that in the ordinary course of things the floors in such a building are never loaded to anything like this amount; but at any time circumstances may cause people to crowd into any room of such a building sufficient to load it with more than twice the amount of 60 lbs. per sq. ft., specified in the Building Law. It is this load, which may possibly occur at any time, that the floor should be designed to safely carry, and not its normal and usual load of only a few pounds per square foot.

Besides this, while a building may start in life as a private residence, changes in later years may put it to entirely different uses which may very likely cause its floors to be frequently crowded with people or loaded with goods or vibrating machinery. This should be borne in mind in proportioning the floor of a structure.

In fairness to Mr. Aus, we should add that with his suggestion to reduce the live load per square foot for which the floor joists are proportioned, he couples the proviso that the floor should be designed to carry safely such concentrated loads as safes and pianos. If this is done it will usually be found that the joists will safely carry the unit loads specified by the Building Law.

Regarding the weight per square foot for which floors should be proportioned, we would call special attention to the recent experiments at Har-

vard University, described by Prof. L. J. Johnson in this issue. These experiments appear to establish the fact that a dense crowd of people may bring a load on a floor as great as 160 lbs. per sq. ft.

Eleven years ago, in our issue of March 16, 1893, we described similar experiments by Prof. W. C. Kernot, of Melbourne University, which were not carried so far as Prof. Johnson's but in which actual floor loads of 135 lbs. per sq. ft. were recorded.

It may be said with truth that such dense crowding as was carried out in these experiments is rare; but on the other hand, it is a fact that curiosity or panic may at any time cause people to crowd any space to the limit of endurance, and it is the engineer's business to provide for this maximum load even though it may never occur. Of course, if this maximum load is assumed, it will be permissible to use a very high unit stress for the material, because the loading is so far above that to which the material will ordinarily be subjected.

STRUCTURAL ENGINEERING IN APARTMENT-HOUSE CONSTRUCTION.

The construction of the lightest class of city tier buildings, of which apartment houses may be taken as a type, appears to be in a condition closely similar to that of highway bridge building during its most evil days. What is primarily a problem deserving treatment by a competent structural engineer is handled without engineering advice, solved by rules and tables, or in very many cases is even left wholly to the prospective contractor. The results are a deterioration in the art, a neglect of detail, and in general flimsy and unsafe construction.

The recent disastrous collapse of the Darlington (apartment) building in New York City revealed some rather surprising circumstances in the history of that structure, the leading features of which were reviewed editorially in our issue of March 24. But it is a fact that many of the objectionable conditions shown to exist are not peculiar to that single case; they are fairly general, indeed, almost typical of that class of construction. As warrant for this assertion we need go no farther, for the moment, than to cite a published statement made by a prominent official of the New York City building department shortly after the Darlington failure. He said that during the past few years scores of apartment buildings had been erected under conditions precisely as dangerous as those which characterized the Darlington. It may be added that structural engineers familiar with current practice acknowledge this statement to be substantially correct.

Current engineering practice in bridge construction, in mill-building work, and in that class of tier-building work which is exemplified in steel cage office buildings, is in thoroughly sound condition and has made material advances within the last few years. In the special field of very light tier-building work, however, the contrary is true. Here we find no comprehensive engineering which considers the individual conditions of each case and adapts its structures to meet the conditions. Instead, the so-called design is made in a most perfunctory manner, and practically amounts to little more than writing down sizes of columns and beams from tables. Detailing is done with a large disregard of the stresses to be met, and any special conditions, such as may arise with increasing height of buildings, are neglected. Practically, we find four-story methods of designing applied to twelve-story problems.

Why is this so? Why is structural engineering not equally efficient in its application to apartment-house construction as it is when applied to bridges or to the more closely related office building? We have already likened apartment-house construction to the highway-bridge construction of a number of years ago. If the results apparent in the two cases are comparable, it is also true that the antecedent conditions are not unlike. In those days of shoddy highway bridges, expense was the only factor considered. Engineering advice or supervision was absent, except such as was given by the prospective contractors. The system under which bridge contracts were let conduced in every way to "skinned" design, neglected detail and generally to the taking of long chances. Such a system inevitably works toward retrogression; the pressure upon the more conscientious bidder to cheapen his work is enormous, and naturally there is no lack of less scrupulous competitors. Substantially similar conditions are found in apartment-house work to-day. Here again expense is the prime consideration, and the consulting engineer is an unknown luxury. When he comes to the structural features, the architect mechanically elaborates a design on the specifications provided by the local building regulations, which are assumed to furnish a sufficient guide to proper construction.* Then the prospective contractors are allowed, or even invited, to modify the plans if they can reduce the cost.† In every way a premium is put upon cheap construction. The building laws are even felt to be a burden, and evasions of their spirit, even where the letter is not infringed upon, are freely encouraged.

A bridge, or the structural frame of a high office building, is appreciated to be a problem demanding skilled and careful engineering consideration, and in consequence it generally secures such consideration. The structural problem of the apartment house is not thus appreciated. On the contrary, those who direct the construction appear to regard it no more highly than a problem in arithmetic, to be solved by simple rules which anyone may handle; a problem, indeed, which might have a much cheaper answer were it not for oppressive restrictions imposed by the municipal authorities.

Here lies the root of the matter: the structural problem offered by a structure like a twelve-story apartment-house is held in far too light regard to produce good engineering. This has come about partly because the problem is usually easier of solution than office-building problems, partly because the money involved is considerably less, and partly because the industry of building apartment houses is mainly a speculative one. Like Hodge's razors, which wouldn't shave, buildings are put up to sell. We can hardly hope for improvement, however, through change in these causatory conditions. Improvement can be attained only through a general appreciation of the fact that the apartment-house structure no less than the office building should have competent engineering work in its design. The collapse of the Darlington building has been an object lesson to the whole building trade. But for results to be of value it is necessary that architects, engineers and all who are concerned with light tier-building work should recognize fully the present condition of this field of work and help to place it upon a higher plane.

It is necessary also that weak and defective methods of construction which have come in during the rush to cheapen work should be plainly recognized and by general consent abandoned.

The use of cast-iron columns for buildings of considerable height is a practice which was retained from the practice in low buildings of large natural stability. Good engineering has tended away from cast-iron columns during some years past, even for low buildings, and their use for buildings higher than five or six stories has been recognized by nearly all competent structural engineers to be doubtful practice at best. Yet they have been very generally used in light tier-building work for much greater heights, and in apartment houses their use has been almost universal. Now that the Darlington disaster has given a sudden and terrible demonstration of their dangerous untrustworthiness, the time is ripe for a change. Cast-iron columns should be entirely abandoned for all buildings higher than five or six stories, say 75 to 85 ft., and within this height they should be restricted to buildings whose width is not less than half their height. The object of this limitation is on the one hand to avoid piling up a vertical series of loose joints to a greater number than five or six, and on the other hand to secure a sufficient number of joints in series horizontally to safely provide for lateral stresses; single-bent construction should be wholly avoided. Some of the objections to cast column construction—the necessarily loose joints and the unavoidably eccentric loading—were referred to in our discussion of the Darlington case in our issue of March 24. The brittleness and unreliable character of the metal furnish equally strong reasons. All of these reasons, moreover, are thoroughly well known to structural engineers.

A no less important point, also referred to in our issue of March 24, is that "shoddy" steel construction may be fully as weak and dangerous as a cast-column construction. Of course, cast-iron columns, when adopted, are used because they are a little cheaper than steel column construction; but it is a curious fact that, since a limiting minimum size for cast-iron columns is often specified (as in the New York Building Code, 5 × ¾-in.), a point of light loading may be reached where columns of simple steel shapes are cheaper than the minimum cast-iron column. In such cases I-beams, channels (with or without stiffening angles), two-angle sections, and even single angles, have been used. Few of these shapes, however, lend themselves readily to satisfactory detailing when used as columns. Generally no special attempt is made to secure good details, but the simplest connections, proportioned only for rivet shear and bearing, without regard to such refinements as eccentric loading, are put in. The result is a building whose lateral rigidity is as low as if cast columns were used, while the lower margin of safety and the greater "slenderness" of the columns make it even less desirable. Construction of this kind should be prevented, first, by insisting on column sections which permit making substantial central connections, and, second, by insisting that such connections be actually provided.

Steel construction of the character referred to is an outgrowth, in part, of the general practice of using an unbraced framework, a practice which is universal in light tier-building work. The frame is charged with resisting vertical loads (except the weight of walls), while the floor filling and the exterior walls are relied upon for stiffening. When everything is in place, and the building is completed, the functions will be properly distributed in the manner intended so long as no settlement or distortion takes place. But whether, if the building as a whole is properly considered as a structural problem, the engineer will be satisfied to design only for this condition is another question. Certain it is that during erection the complete iron frame often must stand alone for a time, held by such temporary bracing as may be put in by the contractor to make the structure plumb in the absence of walls. With cast-iron columns it is difficult or impossible to provide permanent bracing in the frame. But if we abandon these in favor of steel columns in high buildings, as above recommended, we find that the steel frame may easily and at little additional cost be made rigid, not alone rigid enough to stand alone, but sufficiently strong to resist all lateral stresses on the completed building if the stiffening effect of walls be altogether disregarded. Full diagonals are not necessary for this; even knee-braced connections are not required; one of the largest office buildings now being erected in New York City is designed without either diagonals or knee-braces in the frame, the full wind moment for a 30-lb. wind pressure being resisted by the riveted straight connections between girders and columns.

While the structural problem of the apartment house is under consideration, we would ask consideration also for the class of construction known as curtain-wall construction, which is almost universal in apartment-house work. In this type the exterior walls are self-supporting, whereas in full steel-cage construction the walls are carried by beams attached to the framework. The beams nearest the walls are therefore kept clear of the walls, but the columns are enclosed in the walls, often centrally, in order to secure a sufficient thickness of brickwork around the columns (for fireproofing, as required by most building laws) without projections on the inside of the wall. The result of this arrangement is that the details at the walls are badly eccentric, no matter whether steel or cast-iron columns be used. The New York Building Code, as written, especially favors this

*Our position in opposition to such a view of building regulations has already been expressed; see editorial remarks in our issue of March 17 and 24, 1904.

†This practice is far from unusual. A very prominent architect is known to have accompanied the plans for a costly structure of the light tier building type with a note to prospective bidders that any "suggestions tending to reduce the cost of the structure will be considered." That this did not refer to the ornamental or architectural features of the structure may be taken for granted.

class of construction by allowing only a vanishingly small difference in thickness between "curtain walls" and the enclosure walls of steel-cage construction. There is some reason to believe, however, that the modern "curtain-wall" construction was not contemplated when that code was framed, but that the clause specifying the thickness of "curtain walls built between the columns and not carried on the framework" referred to interior walls. Whether or not this be true, there is room for improvement here, in the Building Code as well as in the apartment-house practice. The specified thickness for "enclosure walls" should be reduced to a reasonable limit, thereby putting curtain-wall construction at a disadvantage. Where the latter type is used, however, the ingenuity of the designer should be exercised to the end of securing central connections of the framework.

Unsatisfactory detailing in city building work is materially favored by the general absence of a requirement, on the part of municipal building departments, that detail drawings be filed in addition to the general plans of a structure. It is possible for the design of a structure to be quite satisfactory as far as the general plans, showing the dimensions of main parts, indicate, and yet be objectionable as a whole because of bad details. If it were required that either complete detail plans or sketches of typical details be submitted to the Building Department for approval and file, this possibility would cease to be a source of danger. Such a plan would have the further advantage that it would give a permanent public record of the actual detail construction used in any given building, a record which at present does not exist.

LETTERS TO THE EDITOR.

Recording the Location of Sewer House Connections.

Sir: In recent numbers of your paper I have observed several letters as to the best methods of making the positions of sewer junctions. Possibly a description of the mode adopted here, when constructing the sewerage system, 1890 to 1893, may prove of interest to some of your readers.

Plans on a scale of 50 ft. to the inch were made, on which were platted ground plans of all buildings (from the insurance plans published by Mr. Goad) the alinements of the sewers, with the exact position of all manholes, ventilators, flush tanks, and junctions, with the measurement of the latter from some fixed point, such as a ventilator cover, also at each junction a piece of scantling was brought nearly up to the surface.

These plans also show the position of the house pipes, and the points at which they enter the buildings to connect with the fixtures.

The nature of the subsoil whether earth, hardpan, or rock is also shown on the plans.

Your obedient servant,

Edward Mohun, M. Am. Soc. C. E.

Victoria, B. C., Mar. 30, 1904.

The York, Pa., Sewer Specifications Again.

Sir: I have read with interest the reply of Messrs. Hering and Fuller, of New York, to my criticism of their specifications for the sewer system at York, Pa., in your issue of March 31. Paragraph No. 7 in their letter gives the impression that I made an incorrect and misleading copy of their plan, by stating that, in the sketch presented with my letter, I left off "one sewer with a controlling grade." Were it not for this personal charge I would not have considered a reply necessary on my part, as I do not desire to prolong the discussion of the York sewer specifications. But I cannot allow to go unchallenged a charge of this kind. No sewer, the construction of which was contemplated, was omitted by me, nor was any sewer either contemplated or projected for future construction with a controlling grade, omitted, as an examination of the complete map will demonstrate. The sketch exactly reproduced all sewers covered by the contract.

In the same paragraph they also say: "One street elevation was accidentally omitted on the plan." To have singled out the one omission on their plans would have been frivolous on my part. As a matter of fact the omissions are numerous on the plans supplied to the contractors.

Concerning 30-ft. cuts, of which Messrs. Hering and Fuller state there are none, the plan in my possession shows that the elevation of the surface of the street at the corner of Locust St. and Sheridan Ave. is 405.3, while the elevation of the sewer is 376.5. Rock will be encountered at this depth, so that 0.6 ft. must be added, making the exact depth of cutting 29.3 ft. This is not far from a 30-ft. cut, but Messrs. Hering and Fuller are right, "there are no 30-ft. cuts." Yours truly,

Alexander Potter.

143 Liberty St., New York City, April 2, 1904.

Elevator Car-Safety Appliances and the Building Code.

Sir: The many recent elevator accidents reported in the daily papers have tempted me to write this letter for publication, with the hope that it will attract the attention of engineers, architects and owners of buildings, so that they will unite and use their influence to impress upon the New York Building Department the necessity of revising and enlarging the laws so that only safe elevator machinery can be installed. Under the present framing of the law, safety appliances on elevators have to be approved by the Building Inspectors, but many of those installed are fatally dangerous.

The public cannot expect anything better, because of the inadequacy of the sections relating to elevator construction in the present Building Code.

A careful perusal of the regulations will show that only Section 12 relates in any way to elevator safeties, and reads as follows: "Section 12. All elevators shall have a governor, or speed safety, properly connected to the safety device on the car." This law is absolutely inadequate to secure safe construction, especially when one considers the important detail in elevator construction which it is supposed to cover. The specifications referring to the safety device should be so clearly defined as to require a device to positively stop a car of any load at any speed upon steel guide rails, without such shock or jar as to cause danger to the passengers.

It should be the duty of the Bureau of Buildings to see that every elevator car is provided with a safety device which will comply with such a specification, and the laws should be framed so as to make it impossible for anything but such a safety to be supplied.

Is it a discrimination against any elevator builder for the Bureau of Buildings to rule that certain essential conditions shall exist, provided these conditions can be met by apparatus which is on the open market? The essential conditions of an elevator car safety are:

(1) To stop a car when its normal speed is exceeded.

(2) To begin to check the speed in as short a distance as possible.

(3) To take sufficient distance in coming to a final stop to avoid injury, or even discomfort, to the passengers.

(4) To provide absolute power to stop any load that the car can lift.

The detailed construction is a matter for any building department inspector to keep up to the highest standard of the art, but the essential conditions whereby alone the safety of elevators can be assured should be enacted as a law that must be enforced under all circumstances, and might read about as follows:

All elevators running on steel guide rails shall be provided with a safety device operated by centrifugal speed governor (located overhead), whereby upon excess of normal speed the governor grips and positively stops the safety rope, and whereby the stopping of the safety rope brings the gripping jaws on the car in contact with the rails in not over two feet distance of car travel, and whereby the further travel of the car up to a distance of six feet steadily increases the pressure of the jaws on the rails until the car is brought to a safe and easy stop. No safety device to be accepted by the Building Department that has not fulfilled these conditions on free fall tests, with each gripping jaw set at a running clearance of at least 3-16-in. from the rail, before the safety operates.

The foregoing merely recites the broad functions that are absolutely essential for the safety of all elevator cars running on steel guide rails, and what has been already accomplished in actual practice, and does not restrict detailed construction in obtaining these results; but the enforcement of the law here suggested will secure absolute safety of every elevator, and prevent the general use of devices that are a daily menace to life and limb of millions of elevator passengers.

In view of the large percentage of inadequate safeties on passenger elevators now in operation, this law should be made retroactive, as it is fully as important to have the elevators now in operation made safe as it is for those that are about to be newly installed.

Respectfully submitted,

Edwd. Van Winkle, Consulting Engineer.

100 Fifth Ave., New York City, April 8, 1904.

Horizontal Reinforcement vs. Trussed Reinforcement of Concrete Beams.

Sir: The writer has followed the various discussions on reinforced concrete in the Engineering News with great interest. These seem to concern the relative efficiency of horizontal reinforcement with plain or deformed rods, and the half truss imbedded in concrete.

After some investigation of tests and calculations, the writer has been able to make the accompanying diagram, which bears upon this question, and which he believes, is correct. The results seem to be so marvellous that he scarcely trusts his own judgment on this matter, and invites criticism.

Where concrete beams are reinforced horizontally only, the horizontal shear, or slipping tendency, is transmitted from the bar into the concrete, either by direct adhesion, or by so-called mechanical bond. It is a well known fact that the increment of vertical shear in a beam must be transmitted into the tensile and compressive members within a distance equal to that between their centers, or, in other words, that the vertical shear at the ends of a beam must be transmitted into the steel of a reinforced concrete beam within a distance equal to that from the center of the steel reinforcement to the center of pressure.

Let d = distance from the center of tension to the center of compression.
g = the grip due to adhesion, or mechanical bond, per linear inch.
Then gd = total grip for length d = total vertical shear, and the superimposed load limited to 2 gd.

This value, it will be noticed, is independent of span up to a point where the bending effect is greater than the horizontal shearing effect, and the curve representing safe loading is therefore a straight line to this point. The writer has found that when the span is about 22 times the depth, ultimate adhesion of concrete to steel being assumed at 1,000 lbs. per lin. in. of bond, then the beam should not fail by shear, providing that the concrete be of the very best mixture. If this be the case,

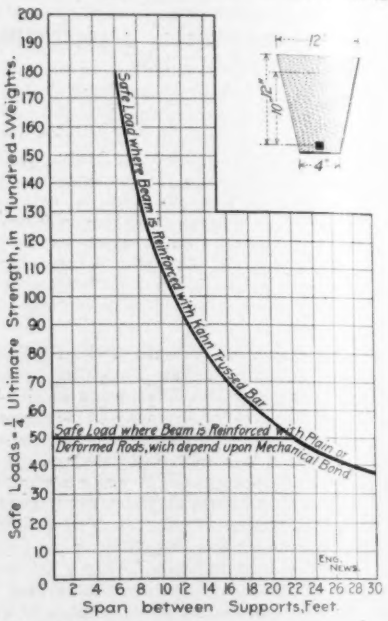

Safe Loads - ¼ Ultimate Strength, in Hundred-Weights.
Span between Supports, Feet
Safe Load when Beam is Reinforced with Kahn Trussed Bar
Safe Load where Beam is Reinforced with Plain or Deformed Rods, which depend upon Mechanical Bond
12"
22"
4"
ENG. NEWS

Diagram Showing Comparative Strength of Kahn Bars and Plain or Deformed Rods of 1 sq. in. Section.

(Distance between center of compression and center of tension, 10 ins.)

then it is possible to develop the full strength of the steel, only when the ratio of span to d (distance between the centers of tension and compression) is 22 or more. However, even if this ratio is kept, the beam will not necessarily fail by breaking of the steel, if high carbon bars or exceptionally large amounts of steel are used. It will fail by compression owing to the limited depth, and therefore, limited compressive area demanded by the ratio given above. The question of course remains open as to whether any engineer would care to trust a construction which had not been reinforced vertically for shear, knowing that if the material is not perfectly mixed, and bond between the steel and concrete absolutely assured, the beam is almost certain to fail suddenly when tested to ultimate destruction.

In computing the case represented in the diagram, the ultimate adhesive value per linear inch of steel was assumed at 1,000 lbs. This is an average derived from tests

RESULTS OF TESTS OF UNION BETWEEN CONCRETE AND STEEL, MADE AT MASSACHUSETTS INSTITUTE OF TECHNOLOGY.

Type of rod and size.	Size of Concrete Block ins.	Length of rod imbedded in concrete ins.	Breaking load in pounds	Adhesion per linear inch, lbs.
Ransome, ½.....	6 × 6	12	12,100	1,008
Johnson, ½.....	6 × 6	12	12,200	1,017
Johnson, ½.....	6 × 6	16	13,120	820
Thatcher, ¾.....	8 × 8	20	21,150	1,058
Johnson, ¾.....	8 × 8	20	27,600	1,380
Ransome, ¾.....	8 × 8	24	31,900	1,329
Thatcher, ¾.....	8 × 8	24	18,300	762
Johnson, ¾.....	8 × 8	24	25,000	1,042
Johnson, ¾.....	8 × 8	36	28,000	778
¾-in. square	8 × 8	24	19,700	821
2¼ × ¼-in.	8 × 8	31	25,500	826
Ransome, 1⅛ ...	10 × 10	27	33,100	1,226
Thatcher, 1⅛ ...	10 × 10	27	28,100	1,041
Johnson, 1¼	10 × 10	37	44,200	1,195
Ransome, 1⅛ ...	10 × 10	50	34,600	692
Thatcher, 1¼ ...	10 × 10	50	58,450	1,169
Johnson, 1¼	10 × 10	50	57,250	1,145

NOTE.—In each of above tests, the rod slipped or the concrete split.

made at the Massachusetts Institute of Technology (Railroad Gazette, Sept. 18, 1903), but is altogether too high to be used in practice, as the mixture of concrete is such a variable factor. It is also quite a different matter whether a bar be pulled through the center of a block of concrete, as was done in these tests, or from one edge of the concrete, as would occur in a beam in actual practice.

It is also interesting to note from these tests a few of which are mentioned in the table herewith given, that the linear adhesion of a rod 1¼ ins. in diameter, is very little, if at all, greater than that of a ½-in. rod. It is wrong to speak of the adhesion of concrete to steel as so much per square inch of cross section of the bar. That does not at all concern the engineer. The basis upon which he works is the linear adhesion per inch of bar. If it is true that this linear adhesion is not greater for a large bar than for a small bar, then a beam with a ½-in. rod will carry as much weight on short spans as one with a 1¼-in. rod.

In regard to the Kahn trussed bar. The important fact noted in the discussion and which seems to have been verified by actual tests, is that the ultimate capacity of the steel was invariably taxed. Thus it may be safely calculated by any of the ordinary formulas, which depend upon the breaking strength of the steel. The curve on the diagram submitted herewith was based upon the assumption that the steel would fail at the center. Safe loads are therefore ordinates to the parabola.

If horizontal reinforcement is used only, the beam should fail very suddenly by shear at or near the elastic limit of the steel. At this time the bar begins to stretch seriously, the tensile strength of the concrete itself is lost, and the adhesive stress is suddenly taxed at the ends, bringing on, invariably, collapse of the concrete beam as a whole, or the familiar failure by horizontal shear. The diagram for horizontal reinforcement only has been plotted on the foregoing assumption. If a beam fails by horizontal shear, the safe load causing such failure is constant, irrespective of the span, as the shearing strength is only dependent upon the weight imposed, not span. The curve for such safe loading would therefore be a straight line up to a point where it intersected with the parabola diagram, or, in other words, where the spans became so great that bending moment played a larger factor than shear.

Maurice Goldenberg.

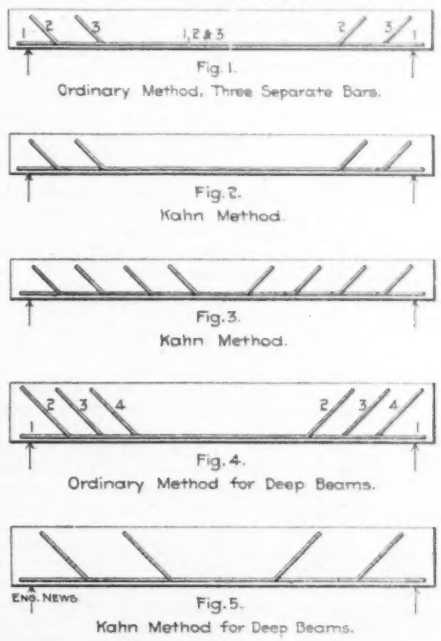
East Boston, Mass., March 24, 1904.

Shearing Stress in Concrete-Steel Beams.

Sir: Mr. Kahn makes so many misleading statements in his letter, published in Engineering News, of Feb. 18, that the writer finds it necessary to reply again. Mr. Kahn's Figs. 2 and 3 purport to give a comparison between the writer's method of reinforcement and his own. To make a strong showing for his own form, Mr. Kahn "o'erleaps himself." He has taken a beam of such length in proportion to its height that it needs no provision against shear whatever. It is well to note, however, that in such a beam as he has shown, the bending moment on the beam between the points a and b, in his Fig. 3, is almost as much as at the center, and hence the area of the wings can not be included in the available section of bar to resist the moment. When sheared up in this way from end to end, practically, there is of course no space of plain bar in the center of the beam, and consequently no danger of slipping, but there is the loss in section above referred to, which is not necessary in what Mr. Kahn calls the writer's method. This method is shown in comparison with Mr. Kahn's in Figs. 1, 2 and 3.

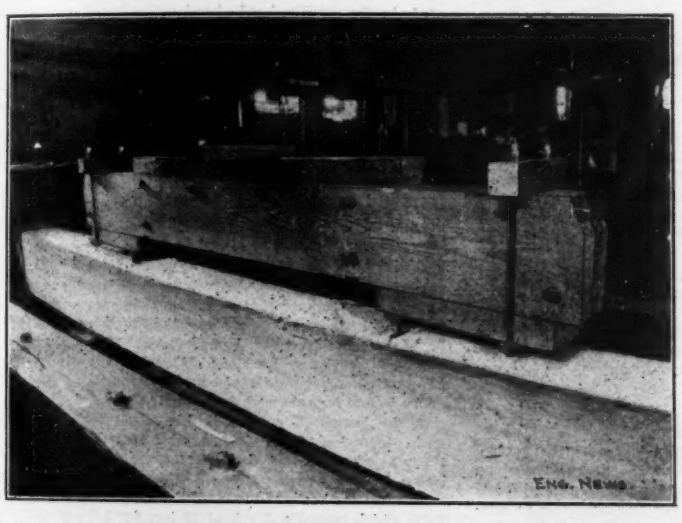
In the first three cases above shown, there will be a certain quantity of metal required at the middle for the maximum moment, supposing that it all has the same elastic limit strength per square inch. In Mr. Kahn's design, Fig. 2, the full bar could be used provided he got a good bonding of the concrete around the bar, which remains intact. He says he does not really need to have the bar embedded at all, as a matter of fact, but in any case only enough to preserve the metal from rusting. But to preserve the metal it is necessary to have it well distributed through the stretching concrete; otherwise the concrete will not stretch, but crack and expose the metal. Covering a large bar with just a skin coat of concrete is not proper dissemination of the metal, and the film would quickly crack; so that the larger the bar, the larger the area of concrete immediately around the bar required. But large bars and large areas do not work as well as small bars and small areas, and this is a distinct disadvantage in Mr. Kahn's bar, where, at the section undergoing the maximum stretch, he has three bars in one, so to speak. It is much better to make the three pieces entirely independent bars, as shown in Fig. 1.

It is also necessary that the bond between the concrete and the metal should remain intact. However absurd a doubt of this may seem to Mr. Kahn, the writer can show him practical examples of failure of such bond in lengths as short as 3 ft. So far as foreign work is concerned, where only plain material has been used, the writer wishes it to be distinctly understood that the success of such construction has not been demonstrated. It is only within the past eight years that they have been using concrete-steel beam constructions wherein absolute unity of action between the concrete and metal was essential; and in the writer's experience, it has required eight years' hard service to develop this weakness. The oldest concrete-steel company, the Monier, originally confined their construction to circular tanks, etc. But for beam construction, they now endeavor to use a bonded netting of some kind, such as welded wire, for example, which is a perfectly scientific material if properly proportioned. In one of the last copies of "Beton und Eisen" there was an article by a Frenchman, stating that it had been shown that it was not wise to depend upon the adhesion, and it is evident that foreign practice will soon change with regard to the use of plain rods.



Mr. Kahn's discussion of the shearing stresses by diagrams 7 and 8 is based in the first place upon a wrong conception of the writer's method of construction; and even if it were not, it is without scientific meaning. The theory of internal stresses, which he quotes, is old and well known, and nothing the writer has said is at all in conflict therewith, though the theory is far from being rigidly applicable to a concrete-steel beam, as it assumes a perfectly elastic body, and neglects the friction of the molecules. The planes of principal stress are therefore inclined much less than 45° with the vertical, and this is borne out in practice, the bottom portion of a shearing break, or after it leaves the horizontal if such crack exists, being usually nearly vertical, and then curving off at the top to a slope of about 45°.

Mr. Kahn admits he made a slight error in the shear load capacity of beam assumed in his previous communi-



(1) BEAM 12 INS. WIDE, 19 INS. DEEP, AND 18-FT. SPAN, REINFORCED BY FOUR ¾-IN. CORRUGATED BARS.

(No reinforcement whatever against shearing.)

(2) Beam 12 Ins. Wide, 5 Ins. Deep and 10-Ft. Span, Reinforced by Two ½-In. Horizontal Rods.

(3) Beam 12 Ins. Wide, 9½ Ins. Deep and 15-Ft. Span, Reinforced by Four ½-In. Corrugated Bars Running Horizontally and by Four ½-In. Vertical Bars at Each End.

FIG. 6. VIEWS OF CONCRETE-STEEL BEAMS TESTED AT TERRE HAUTE, IND., SHOWING FAILURE BY RUPTURE IN COMPRESSION.

cation, resulting in getting this only one-half what it should have been, but disputes the writer's statement that he had given the moment capacity as twice what it should have been, saying that he would not hesitate to load one of his beams having twice the metal in it to twice the amount given by the writer's published diagrams.

The writer herewith presents some cuts of tests recently made at Terre Haute, in which the results were from 2% to 28% above his formula, reinforced with only the quantity of metal called for thereby, and all failing in compression of the concrete on top. These results are not intended to be given to the public in detail until the investigation is complete, but the engravings are shown here to refute Mr. Kahn's statement that corrugated bar beams always fail by shear, and his reference to the tests by some railroad company of twelve beams, all of which failed by shear, concerning which the writer has no information whatever. These beams may have been improperly made, they may have had too much metal reinforcement, may have been loaded at two points too far apart, may have been too high in proportion to the length of span, or may have sheared at practically the ultimate strength in compression. Probably, even as made, if they had been tested with a uniformly distributed load, they would have been found to be entirely satisfactory.

Some of the beams tested at Terre Haute had four ½-in. shear bars set in the beams vertically at each end, without any connection between them and the horizontal reinforcement. Some of the beams had some of the bars brought up diagonally at the ends, but it is interesting to note that just as high results were obtained on all beams without any provision for shear. The beams were loaded at two points, one-third of the span apart.

Now, if the quantity of metal here used was sufficient to cause a compression break, then it is manifestly absurd for Mr. Kahn to suppose that with twice the metal in the tension flange, he or any one else could get twice the capacity of beam. It certainly is true that with twice the quantity of metal, an increase in strength would be obtained on account of the lowering of the neutral axis, but this increase would not be at all proportionate to the extra expense involved, and it would be much cheaper and much better to increase slightly the depth of the beam, and the shearing stresses would at the same time be decreased thereby. With certain prices for the materials assumed, there is undoubtedly a certain ratio of metal to concrete which would represent the most economical construction, and the writer has had occasion to investigate this point, and believes that for general average conditions, the ratio that he has taken is the best; but is perfectly willing to concede that this is a fair point for investigation by the committees of the various societies, appointed, or shortly to be appointed, for the purpose of investigating this subject. As a matter of fact, it takes, theoretically, nearly twice the quantity of metal in soft steel for a beam of given strength that is required where a steel of double the elastic limit is used, though not quite so much concrete.

Mr. Kahn bases his assumption, that he can get twice the strength by using his bars with double the quantity of metal reinforcement, upon results obtained in a test on a section of floor published in the Engineering News of Oct. 17, and also in Mr. Kahn's first catalogue, p. 14, Fig. 8, where he had two beams joined together at the top by a flat floor plate. To attempt to present results on such T-beam construction for comparison with tests on beams of rectangular section is essentially misleading. An exposition of the characteristics of such beams, and a method for proportioning same for given loads—which method has been checked by results in actual practice—is given in the writer's catalogue, pp. 57-59.

The above is about on a par with Mr. Kahn's comparison between the results obtained in floor construction in Washington with the construction mentioned by the writer in his first letter. The writer stated that a cinder concrete floor 6 ins. thick, with 0.3 sq. ins. of metal per foot of width, would carry 615 lbs. per sq. ft. ultimate load on a 12-ft. span. Mr. Kahn comes back with the statement that on a 16-ft. span, with concrete 6 ins. thick, but with 0.6 sq. ins. of metal per foot of width, he carried "52,000 lbs. on 62 sq. ft., equal to 840 lbs. per sq. ft., without the appreciable deflection of a hair." Now, we would like to know, (1) what kind of concrete was used; (2) what was the shape of the area on which he applied the load; (3) was that area detached before the test from the adjacent floor; (4) what material was used in loading, and did it have opportunity to arch; (5) what size hairs were used in measuring the deflection.

When we get this information we will be able to figure out the equivalent uniformly distributed load, and how well or how ill it fulfilled reasonable expectation. In this connection, the writer might state that Mr. Kahn's tables, as published in his new catalogue B, give for 150 lbs. safe load on 16-ft. span, 1:2:5 rock concrete, the same thickness of concrete and about the same quantity of metal, as do the writer's tables. Mr. Kahn's tables are based upon a crushing strength of concrete of 3,000 lbs. per sq. in., while the writer's are based upon 2,000 lbs. per sq. in. A strength of 3,000 lbs. per sq. in. at 28 days is much more than should be counted on. The reason for the similarity in result obtained is that this erroneous assumption of strength is offset by an equally erroneous method of computing the strength of a concrete-steel beam. Mr. Kahn assumes a continuous rectilinear relation between stress and strain, which is, of course, nowhere near the truth. Furthermore, he assumes a fixed position for the neutral axis, whereas if he will make some actual tests on rectangular beams and measure the extension on the beam itself at various planes, it will be very quickly demonstrated to him that this neutral axis, while originally starting at the center of gravity of the section transformed into its equivalent in concrete—which is where he assumes that it stays throughout the period of the test—rapidly rises to a point considerably above the center of the beam, depending, somewhat, of course, upon the quantity of metal used, and then remains practically stationary at this high elevation until failure occurs.

Mr. Kahn's tests, so far, have been made by himself. We suggest that he have some made at some of the universities, where all of these matters can be properly and thoroughly investigated, and we think he would then soon learn that there would be found other and simpler ways to obtain just as good, if not better, results than could be furnished by his bar. These tests, however, should be made with a uniformly distributed load, as this is representative of 90% of the conditions in actual practice. It would then be found that the shear is not, after all, the serious matter that Mr. Kahn would have us believe, because, as a matter of fact, the maximum shearing stress in a concrete-steel beam seldom amounts to anything serious until cracks have developed in the tension flange so close to the end support as to give too small a shearing area in the plane immediately above the metal reinforcement, from the crack to the end of the beam. At this time the stress in the bars is probably very great, and the beam is carrying several times the maximum safe load in actual practice, and a failure by shear, even if it did occur, might be within 10% or 15% of the theoretical strength of the beam. In most cases in practice it would probably be within 5% or 10% of the actual strength of the beam, the load being uniformly distributed, and the maximum moment, where the first cracks would occur, being only at the middle. In laboratory tests, it is generally desired to have a section of uniform bending moment in the middle of the beam, but in such cases the crack is just as apt to occur under one of the loading points as in the center of the beam, and this will at once produce heavy shearing stresses in the plane immediately above that of the metal reinforcement, between the loading point and the end of the beam. Especially is this the case since the maximum shearing stress at or near the loading point is likely to be double the average shearing stress. On the other hand, unless a considerable distance is used between the loading points, the shear on the vertical section will be considerably less than it would be for a uniformly distributed load, which would be unfair also. An apparatus for testing rectangular beams with uniformly distributed load, by means of hydraulic pressure, having 100,000 lbs. capacity, with a span of 16 ft., would not cost over $500, and the writer thinks that all universities ought to have such an equipment. In conclusion, the writer will say as follows:

(1) For a given strength the Kahn bar will require a relatively large quantity of metal, both because of its necessarily low elastic limit, and because of the natural waste in available section for moment, where sheared up from end to end as it should be to give a good bond throughout, and smaller areas for the concrete to surround so as to preserve the stretching quality of the latter, upon which the whole art rests.

(2) Unless the bars are cut up from end to end they would have to be prepared for a particular place, requiring a detail for each bar, which is totally impractical on work of any variety of lengths.

(3) Owing to the method of construction, the larger the shear pieces the further apart must they be; so that less shearing provision can be obtained in deep beams where it is particularly required. Compare Figs. 4 and 5.

None of the above objections apply to the corrugated bar, having an elastic limit of about 55,000 lbs. per sq. in. These bars are delivered straight, and bent cold on the job without difficulty. Only 0.7% metal reinforcement is used for beams instead of 1.3% required for low elastic limit material. Somewhat deeper beams are used with the corrugated bars, but the shearing stresses are less, and the cost much less. Where shearing stresses need to be cared for, it can be easily done by bending up the ends as shown in Figs. 1 and 5, and as shown by latter figure, extra shearing provision can readily be furnished when needed.

Very truly yours,

A. L. Johnson.

St. Louis, Mo., Feb. 23, 1904.

(Proof of the above letter was submitted to Mr. Kahn and his reply is appended.—Ed.)

Sir: The writer stated in his original article (Eng. News, Jan. 21) that for correct reinforcement of concrete, the fundamental principles to be observed are: First, Concrete steel beams should be reinforced both vertically and horizontally; Second, Vertical reinforcement should preferably be set at an angle of 45°; Third, Vertical reinforcement should be rigidly connected to horizontal reinforcement.

He has repeated himself regarding these principles in his further discussion in your issue of Feb. 18, calling attention to the 26 tests made by a most impartial body of men, 24 of which failed by vertical or longitudinal shear. The majority of these were reinforced horizontally only; some, however, with vertical stirrups as recommended by Mr. Johnson in his catalogue of 1903.

In your issue of Mar. 17 Prof. L. J. Johnson, of Harvard, called attention to failures by shear, of beams which he had tested, and concluded with Paul Christophe, that the safe allowable value for concrete in vertical shear should not exceed 14 to 30 lbs.

Now does Mr. A. L. Johnson concur in the importance of the principles mentioned above, or does he disagree? His attitude towards vertical reinforcement leaves one very much in doubt, for he starts out in his discussion showing how he proposes to supply such reinforcement, and ends up by saying that shear is not, after all, a very serious matter.

Now, shear may not be so very important from Mr. Johnson's own standpoint as to what should be called the actual failing point of beams, for in his catalogue he sets this failing point at the elastic limit of the steel or when the first crack appears in the bottom of the concrete beam. A beam reinforced horizontally only, collapses invariably by vertical or longitudinal shear, very shortly after the elastic limit of the steel is reached; as the rupture of the tensile strength of the concrete suddenly throws an increased stress upon the adhesion of the concrete to the metal, causing sudden failure shortly thereafter. Where reinforcement is horizontal only, the writer therefore agrees that the time of failing should be set at the elastic limit of the steel, and he readily accounts for Mr. Johnson's statements regarding the close relationship of actual tests with the formulæ advanced in his catalogue, based on this so-called ultimate strength.

As for the writer's method of reinforcement, every test to destruction that has been made on his system to his knowledge, has developed the ultimate strength of the steel directly at the center. This being the case, he calls the failing point of his beam the time when the steel parts; which, if compression has been properly taken care of, is in the neighborhood of twice that based on the elastic limit of the steel. Where horizontal reinforcement only is used, the factor of safety in a construction should therefore be based on the elastic limit of the steel. Where, however, it is known with positiveness that the steel pulls in two at ultimate destruction, the factor of safety should be based on the ultimate strength of the steel.

As for failure by compression, a construction where this can possibly occur, is dangerous and shows poor design. In the first place, concrete never has the same compressive strength twice in succession; secondly, failure of this kind is sudden and without warning. With the writer's method of reinforcement the beams should preferably be battered as in Fig. 1, and the concrete which is naturally wasted in the bottom of the rectangular beam may be used to assist in compression assuring safety against failure here. A little superfluous area of compression would even be advisable. In general construction, the beam is surmounted by the entire floor slab which produces a top flange almost impossible of compression.

The writer regards with interest Mr. Johnson's statement that the beams illustrated in his recent article were of such proportion of height to length as to really require no shear reinforcement. Let us measure them. (See Eng. News, Feb. 18, 1904, p. 158.) The proportion of height to length in the beams illustrated in Figs. 1 to 5 is about 1:18, which means a depth of 12 ins. for a beam of 18 ft. span, 14½ ins. for a beam of 22 ft. span, and 16 ins. for a beam of 24 ft. span. Are these proportions so abnormal? For ordinary wood construction we would probably use for the given spans beams of such depth as these, yet for reinforced concrete, our correspondent jokes about them and calls them very shallow. Now the fact of the matter is, the writer has tested sufficient beams to warrant him in saying that a reinforced concrete girder, if properly designed, can be built to have the same carrying capacity as a steel beam of the same depth. The writer knows of no special reason, therefore, why concrete girders should necessarily be of excessive depth.

Mr. Johnson claims to improve on the writer's proportions by making them 1:9½ for Figs. 1 to 3, and 1:6½ for his Figs. 4 and 5, which in actual construction would give results about as follows:

Proportion of depth to length:	Span.	Depth.
1:9½	18 ft.	23 "
	22 "	28 "
	24 "	30 "
1:6½	18 "	33 "
	22 "	41 "
	24 "	46 "

It would be interesting to find an architect or engineer who would be willing to have a building put up with beams of such proportions as these. On a 24-ft. span, which is an every-day occurrence in a building, no owner in existence would have so little judgment as to allow the general use of 46-in. beams.

Beams built of proportions about as the writer has indicated, and reinforced horizontally only, invariably fail by shear. The writer has no knowledge of the publication of any test to destruction, other than those already referred to, where, with *uniform loading*, a reasonable quan-

tity of steel, and sufficient concrete to prevent failure by compression, the steel has pulled in two at the center.

Now, the writer does believe that a scientific discussion should be entered into with justice and fairness by both parties. Let us follow in detail our critic's portion of it. He shows in his Figs. 1 and 4 the so-called "ordinary methods" and compares them with the writer's method. These very "ordinary" methods must have been in common use by him, and vertical shear should after all be a very ancient matter to him. Now it will be very interesting to trace the evolution of Mr. Johnson's ideas from his various statements in engineering publications.

The catalogue issued by the Expanded Metal Co., prior to 1903, wherein Mr. Johnson is mentioned as the company's engineer, give formulæ, but shear is not even mentioned. In reply to the article of Captain Sewell, in Engineering News, of Jan. 29, 1903, heretofore referred to by the writer, Mr. Johnson said (Engineering News, March 19, 1903):

> In deep beams, what we call shear bars are used, perhaps ¾-in. square bars passing vertically through the beam, spaced quite close together near the abutments, the space increasing toward the middle of the beam.
>
> . . . . . .
>
> The writer has recently inserted a discussion of the shearing stresses at the end of the beam in a new catalogue that his company has now in press, which will tell when shear bars are necessary, and the statement is made therein that an increase in capacity of 50% can safely be depended upon by their judicious use under such circumstances, but no rule is given practically or theoretically for the determination of the number of shear bars that would be required and the method of arrangement of the same. It would be very difficult indeed to do this, there never has been made any investigation of what might be called stress strain shear diagram for concrete, etc.

From this we are led to infer that in 1903 Mr. Johnson recommended placing occasional vertical bars in his concrete, but practically acknowledges lack of knowledge as to their correct spacing.

In reply to the writer's recent article, Mr. Johnson stated that the turning up of the bars at the quarter points and again midway beween these and the ends, would be quite sufficient. Finally, Mr. Johnson brings forth a system patented in Europe by Locher & Co., which most nearly approaches the writer's method, but which seems to have been dropped there on account of its impractical nature, and he calls this the "ordinary" method.

Now the writer is satisfied that prior to 1903, Mr. Johnson gave shear study very little attention. After Captain Sewell's discussion, he followed the Hennebique plan and stuck loose vertical bars in his concrete; and, finally, after the writer's discussion, he adopts the Locher system and calls it well established practice.

Now, let us examine the Locher system and Mr. Johnson's method of improving it. In the first place, the writer would not construct a beam such as illustrated in any one of the diagrams of Mr. Johnson. In Figs. 1 and 2, entirely too much space is left in the center without vertical reinforcement. The writer prefers to adhere to the so-called abnormal figures of his article in Engineering News, of Feb. 18. In these, if we regard Fig. 3, we find that ten vertical shear members have been provided for on each side of the center of the beam. If this were accomplished by the Locher system, at least 11 bars would have to be required, one bar extending horizontally from support to support. Take another beam, say 15 ins. deep on a 25-ft. span. If shear members were set at distances apart equal to one-half the depth of the girder, as in Fig. 4 of Mr. Johnson, and the middle 5 ft. were left blank, this would require 18 independent bars.

Now the impractical nature of the Locher construction will at once become apparent. First, when one takes into account the large number of beams required in the ordinary construction, and the multiplicity of bars for each; the field work necessary in placing these bars, and bending them to the proper angles; and the increased size of concrete beams in order that all bars are properly surrounded by concrete.

Second. The quantity of metal turned up at various points is altogether too large for the reduction in stress in the bottom chord, the stress in the inclined member being increased by the secant of the angle of inclination.

Third. No unison whatever exists between the various bars, and stress from one can only be transmitted into the others through the layer of concrete surrounding all. The writer has seen beams in buildings wherein the bottom was so cut with steel bars to obtain the required tensile strength, that of its own accord the concrete dropped off below the bars before any load whatever was placed on these beams, thereby practically ruining the beam.

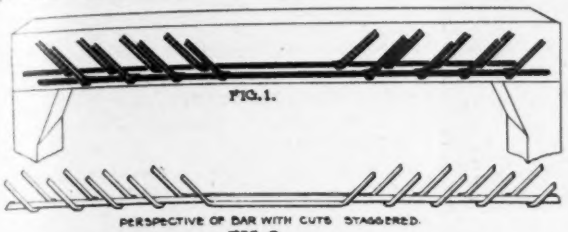
Fourth. For moving loads this system is entirely inadequate on account of lack of unison, as already stated.

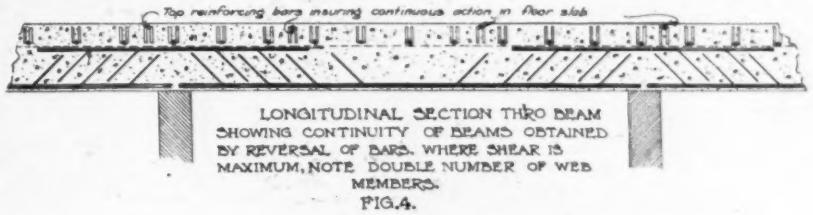
Fifth. If this system had been practical, it would have forced itself into more common use. The fact that it was scarcely known in this country up to within some six months ago, and of its scant practical application in Europe is fairly good evidence that it is too cumbersome, entails too much detail work, and entrusts altogether too much to field work.

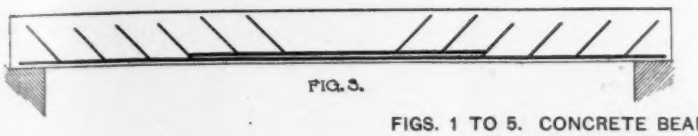
But let us regard Mr. Johnson's improvements on the Locher system. Mr. Johnson recommends corrugated bars. Hennebique, Monier, Considère, Christophe, and other prominent workers in reinforced concrete were entirely satisfied and accomplished wonderful results with plain round rods, but Mr. Johnson insists on having them corrugated. He saves from 4% in Fig. 4 to 6% in Fig. 2 in the length of his bars, but wastes from 8% to 12% of metal in his corrugations. Now, if his corrugations had caused no special loss of metal as is done with the Thatcher or Ransome bar, they might have bettered things. To make up for this loss Mr. Johnson gives us a high carbon steel with an elastic limit of 55,000 lbs. How sad indeed that bridge engineers have not adopted this high grade of steel for constructions in the past, when they might have made a saving of about 40% in all their structures. The writer doubts whether a dozen competent engineers in this country would trust for tensile purposes a high carbon steel bar having an elastic limit of 55,000 lbs. The writer has taken a bar of this nature and in throwing it on a pile of other bars has caused it to snap in two. High carbon steel is unreliable for tensile purposes. It has little resilience; it is brittle and breaks suddenly. Not satisfied with using the bars as they leave the rolls, however, Mr. Johnson insists on giving them a bend of 45°, or completely around the flange of a beam. Now a half inch bar may possibly stand a 90° bending without cracking, but the writer doubts whether a ⅝ in. bar will do it, and a 1 in. or 1¼ in. bar will certainly break.

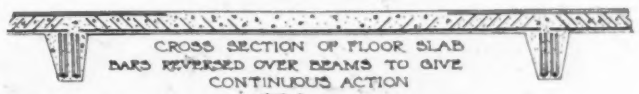
Now as for the Kahn method of reinforcement, it does seem unfair to it that Mr. Johnson, who had probably never seen one of the trussed bars at the time of writing his discussion, should find such serious fault with it. In the first place, the Kahn bars are made of open hearth material with an elastic limit of about 32,000 or 33,000 lbs., which the writer believes about right to assure a good quality of mild steel.

Secondly.—In any beam two or more bars in place of one should preferably be used. This is intended fundamentally as a safeguard against a possible flaw in the steel itself. With two bars the shear members may be staggered with each other, as in Fig. 1, and therefore appear apart at distances only half as great as shown in Mr. Johnson's Fig. 4. Shear members may be comparatively close to the center, as the bottom chord for a beam with, say, two of the writer's 1 in. bars would only be weakened by about one-tenth of the area of the complete bottom chord cross section, instead of one-third, as with Mr. Johnson's Fig. 1, or one-fourth with his Fig. 5.




Top reinforcing bars insuring continuous action in floor slab
LONGITUDINAL SECTION THRO BEAM SHOWING CONTINUITY OF BEAMS OBTAINED BY REVERSAL OF BARS. WHERE SHEAR IS MAXIMUM, NOTE DOUBLE NUMBER OF WEB MEMBERS.
FIG.4.





FIGS. 1 TO 5. CONCRETE BEAMS REINFORCED BY KAHN SYSTEM.
(Diagrams Accompanying Letter of Julius Kahn.)

If, however, Mr. Johnson desires still more shear members, objecting seriously to cutting up two at a time, and if he is prepared to place an order with the writer, the latter will gladly shear these alternately from opposite sides of the bars, as in Fig. 2. As has already been said, every test to destruction which the writer has ever seen on his beams has failed in the steel at the center. Does Mr. Johnson want us to improve on this method of failure?

If the slightest multiplicity of bars is entertained, using trussed bars, a saving of from 20% to 30% can readily be made by arranging them as in Fig. 3, or where it is possible to make them continuous, as in Fig. 4.

Our correspondent claims that a beam reinforced with the trussed bar and having no shear members near the center is necessarily weak there, as the reinforcement at this section is only plain, without the corrugations of his bar. The stress which is carried from the concrete into the steel is always equal to the increment or vertical shear, and this is comparatively small within the middle third of the beam. Take, for example, a case from Mr. Johnson's own tables. He gives for a beam 21 in. wide, 1.8 sq. in. of steel, 21 ins. deep from the center of the steel to the top of the beam, an ultimate moment of resistance of 2,000,000 in.-lbs. For a 24 ft. span with a factor of safety of 4, this would mean a safe load of approximately 14,000 lbs. when equally distributed, or 580 lbs. per ft. of beam. If we were to consider the middle 8 ft. as an independent concrete beam, supported by the end portions and loaded with 580 lbs. per ft., even without steel reinforcement, the tensile fiber strain in the concrete would be only 53 lbs. per sq. in., neglecting all efficiency whatsoever of the steel in this locality. The adhesion of concrete on the steel in the middle third of the beam is, therefore, not so important a matter as our correspondent would have us think.

FIG. 6. TEST OF CONCRETE FLOOR SLAB 10 INS. THICK, 6 FT. WIDE AND 15-FT. SPAN.
Reinforced with Kahn Trussed Bars ¾ x 2 ins. Spaced 14 ins. on Centers. Load, 14 Tons.

Referring again to the tables published in Mr. Johnson's own catalogues, we find in his little red-covered book the accompanying table.

From the results given in this table it would appear that the adhesion per square inch of metal and per linear inch of embedding in concrete for a plain bar would be equally as efficient as a corrugated one.

FIG. 7. APPEARANCE OF FIRST CRACK; LOAD, 44,600 LBS.

Replying to such questions as the writer considers of importance:

The formulas published in the writer's catalogue are not his own. They were advanced by the late Prof. J. B. Johnson, in Engineering News, Jan. 3, 1895, and seem to give results in very close accordance with actual tests.

There is no truth in the statement that the writer assumes a continuous rectilinear relation between stress and strain. In Prof. Johnson's formulas the writer has increased the area of compression to that of a parabola instead of a triangle, to allow for the increased modulus of elasticity and strain, as stress is decreased towards the neutral axis. This is probably as near the truth as anything we know regarding the variability of the modulus of elasticity at the present time.

The writer has made no statement whatever regarding the shifting of the neutral axis. For a given beam safe load calculation, a fixed position of the axis must be assumed. It is, of course, well known that when the elastic limit of the steel is reached the neutral axis suddenly rises.

Tests on the Kahn system have in nearly all cases been made by independent parties, and at the present time there are five or six different universities experimenting on his bar.

Mr. Johnson says:

"As a matter of fact, the maximum shearing stress in a concrete steel beam seldom amounts to anything serious until cracks develop in the tension flange so close to the end support as to give too small a shearing area in the plane immediately above the metal reinforcement from the crack to the end of the beam.

The writer begs to submit herewith several photographs on a recent test of a reinforced concrete floor, having plaster of paris blocks interposed between five small beams 4 ins. by 8 ins., with a 2-in. concrete floor slab on top span 15 ft. (see Figs. 5, 6, 7 and 8). The first crack appeared when the superimposed load was 44,600 lbs., the elastic limit of the steel having been reached at apparently the center of the beam. As the load was increased, other cracks appeared in the concrete at distances increasing from the center, the elastic limit of the steel being arrived at, at various points farther away. The five beams continued to deflect under the increasing loads until the entire series of beams had deflected 20 ins., and touched the ground, the load now being 65,400 lbs. Vertical cracks appeared within the middle 6 ft. only, not a sign of a crack was visible in the beams 4½ ft. from each end.

The cracks at the ends of the beam which Mr. Johnson speaks of must be those which others call shear cracks; for with proper vertical and horizontal reinforcement there should be little occasion for cracks in this locality.

Note the warning failure of such beams as those illustrated in Figs. 6, 7 and 8. If a concrete beam can be built, which, before actual ultimate failure will deflect 20 ins., the engineer is certainly not taking great chances. A steel beam alone would certainly buckle in its top flange when the deflection was probably 4 or 5 ins. It is hard to say just what it would do if properly steadied sideways. But here we have reinforced concrete beams which are capable of deflecting 20 ins. before parting of the steel or crushing of the concrete.

Compare this test with that illustrated in Mr. Johnson's Figs. 6 and 7. The first of these shows a beam 5 ins. deep, 12 ins. wide and 10 ft. span, the second a beam 9½ ins. deep, 12 ins. wide and 15 ft. span. Mr. Johnson does not state the load, otherwise more interesting facts might be developed. In these illustrations note first the proportion of beams; secondly, their method of failure; thirdly, the concentration of loading, which is so contrary to Mr. Johnson's own recommended method of uniform loading. The beams are so wide as compared with the depth that it is hard to conceive their showing vertically.

Type of bar.	Area of cross sec. sq. ins.	Size of concrete block. Ins.	Length of rod imbedded Ins.	Failing load. lbs.	Adhesion per sq. in. of metal per linear inch imbedding. lbs.
Plain 1½ x ⅝	⅝	8x8	24	20,300	1510
" ¾-in sq.....	⅝	8x8	31	22,600	1296
" 1½ x ⅜	⅜	8x8	31	20,300	1164
" 1½ x ½	½	8x8	31	21,700	1244
" 2½ x ½	½	8x8	31	25,500	1461
Corrugated ¾-in....	⅝	8x8	36	28,000	2072
" 1¼-in....	1.07	10x10	27	40,100	1400
" 1¼-in....	1.07	10x10	50	57,250	1070
Thatcher 1¼-in.....	1.00	10x10	50	58,450	1169
" 1¼-in.....	1.00	10x10	37	48,950	1320
Ransome ⅝-in	⅝	8x8	36	36,600	1600

In conclusion, the writer begs to say that with the use of a little judgment and skill a beam can be designed with trussed bars with almost theoretical economy. It may be built with two bars instead of 10 or 12. These bars leave the shop completed units ready for installation, in the same manner as we ship completed latticed girders, in preference to assembling them at the building. For important girders, every bar is marked and cut to length, which eliminates all possibility of error in field work, and this seems to be the only really safe way of working, considering the fact that the average engineer must remain in his office and dictate by letter instead of personally supervising.

Safe reinforced concrete construction demands that important features of construction be eliminated from field work as much as possible, and that the only responsibility placed with the workmen be the proper manufacture of material and its placing.

Julius Kahn.

Detroit, Mich., April 9, 1904.

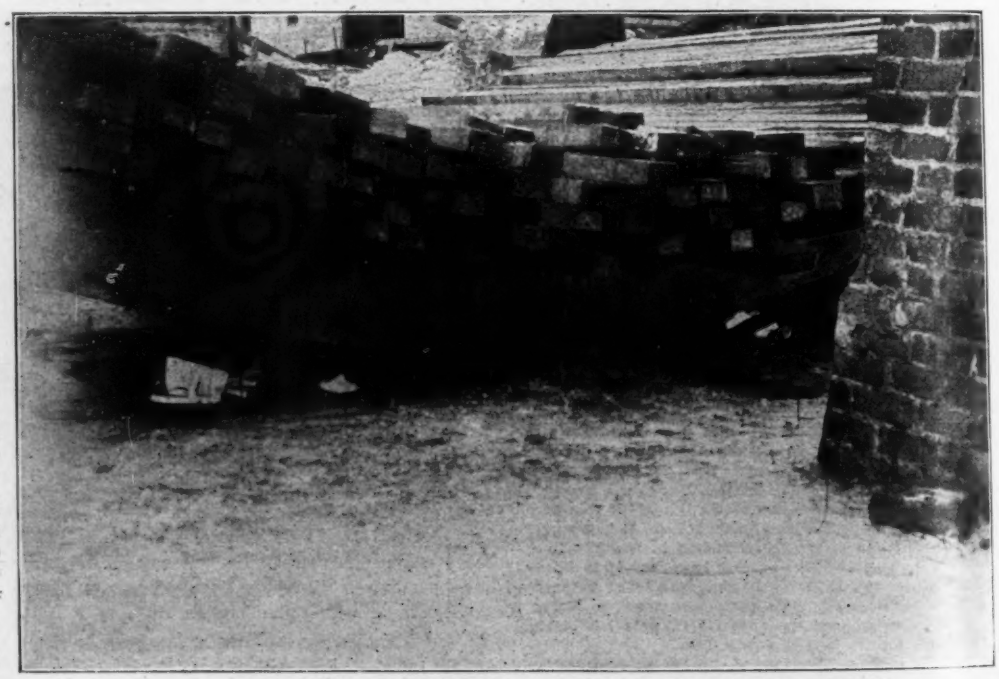


FIG. 8. FLOOR LOADED WITH 65,400 LBS.; DEFLECTION AT CENTER, 20 INS.

TYPHOID PREVALENCE IN MONTREAL

during the past winter is ascribed to pollution of the water supply. The Board of Health of the Province of Quebec has adopted the following resolutions:

(1) That they have every reason to believe that the outbreak of typhoid fever, paratyphoid and diarrhoea among the population of the Montreal suburbs has been caused by the pollution of the water supplied to that population by the aqueduct of the Montreal Water & Power Co.

(2) That in their opinion the intake of that water supply has always been and is still in a bad situation because the water is continually exposed to be polluted in many ways, especially by the sewage that is brought into the St. Lawrence River above the intake, by the River St. Pierre when its waters flow over the dam, by the village of Verdun, by the Verdun Insane Asylum and by the industrial and other establishments situated further up.

(3) That if they had to decide upon the advisability of establishing a new intake at the point where is now situated the intake of the Montreal Water & Power Co. and that such a plan was submitted to them as required by law since 1894, the Board could not do otherwise than to refuse their approval of it.

(4) That as things are actually the question is not of establishing a new intake, but of remedying the bad situation of one already existing, they are of opinion that the present conditions of the intake could be improved by adopting simultaneously the three following measures:

(a) Prevent by some efficient method, dam, channels or otherwise, the intake of the aqueduct from being polluted by the sewage from River St. Pierre, Verdun or other establishments above. Drinking water being concerned, the purification of the sewage previous to its discharge into the river cannot be considered to furnish sufficient protection when the intake of the water supply is so near the discharge of said sewage.

(b) Filter the water so as to purify it in a manner to allow it to be used without danger.

(c) Make sure that the intake pipe, which extends 1,500 ft. out in the channel, is perfectly tight.

(5) That if the Montreal Water & Power Co. cannot carry out the above prescribed means of protection which are absolutely necessary, the Board believes that it would be a matter of public necessity and even in the interest of the company not to persist any longer in maintaining the intake of their aqueduct at the place where it is now situated.

It should be noted that the central part of Montreal, comprising most of the city proper, is supplied by a municipal water-works which obtains water from an open

canal tapping the St. Lawrence River some 3 to 4 miles above the station. The outer part of Montreal is composed of towns or villages which are politically separate from the city. These towns, and also one of the wards (St. Denis) of the city of Montreal, are supplied by the Montreal Water & Power Co., a private company, which takes water from an intake in the St. Lawrence River at the upper (southern) end of the city. A short distance above this intake is the outfall sewer of the town of Verdun, lying just south of Montreal; this outfall was constructed only a few years ago. It is believed that at normal summer stages this sewer does not affect the Montreal Water & Power Co.'s intake, but may do so at low water in early winter. The prevalence of typhoid and intestinal disease above referred to was first noticed in the early part of December, 1903, and continued probably for several months, although in the absence of a thorough system of keeping vital statistics for the Montreal district the information on this point is imperfect.

THE BALDWIN BALANCED COMPOUND LOCOMOtive, built in 1901 for the Plant Lines (Eng. News, March 13, 1902), was not accepted by the railway company owing to its weight being greater than specified, as we are informed by Mr. W. H. Young, Superintendent of Motive Power of the Atlantic Coast Line, which has absorbed the Plant Lines. It has been tried on different roads, and we have the following particulars of its performance from Mr. J. E. O'Hearne, Master Mechanic of the Wheeling & Lake Erie Ry.:

The engine was in use on this road for about six months, during which time it gave satisfactory service in every way, with the exception of the grates and the engine truck. The truck journals were too small for the weight they had to sustain, and would heat at high rates of speed. The boiler had a Vanderbilt firebox, and owing to the small air space under the grates, they would become overheated and were frequently melted, especially when the netting in the front end became clogged. The combustion chamber tube also gave us considerable annoyance, and required to be renewed quite frequently. As far as the design of the machinery of the engine is concerned, it gave us excellent service.

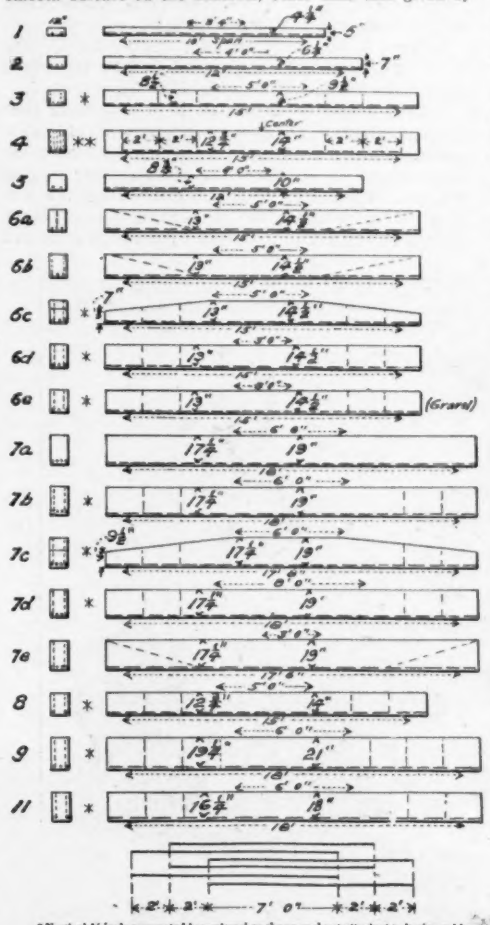
CROSS BENDING TESTS ON CONCRETE-STEEL BEAMS.*

By M. A. Howe,† M. Am. Soc. C. E.

Through the courtesy of the St. Louis Expanded Metal Fireproofing Co. I am able to give you the results of a series of tests recently made at the Rose Polytechnic Institute, under my direction, upon beams of concrete reinforced with corrugated steel bars. The object of the test was to obtain the actual strength in cross-bending of full size concrete beams reinforced with corrugated bars, according to the formula of Mr. A. L. Johnson. It was afterwards decided that, in conjunction with the determination of the cross-bending strength, there should also be recorded the deflections and the movement of the neutral axis.

In order that the concrete and steel should represent products likely to obtain in practice, Atlas cement, bank sand, crushed rock and corrugated steel bars were purchased in the open market. The mixing was done by a local contractor of experience, with his own gang of men and in the manner he ordinarily employed. The cement was satisfactory in every way. The sand, while containing some "dirt" in the form of yellow clay, was a fair representation of bank sand used in Terre Haute. The crushed limestone was in two parts, one coarse and the other fine, the two mixed forming the run of the crusher. The coarse stone would pass a screen of 1½-in. mesh and would stop at one of ½-in. mesh; and the fine stone would all pass a screen of ½-in. mesh. The corrugated steel bars, when tested, showed an elastic limit of about 60,000 lbs., and an ultimate strength of about 100,000 lbs. The areas of the bars were, in general, as advertised, with the exception of the ⅝-in. bars, which were found to be about 20% under size.

The proportion of cement, sand and stone was practically one of cement to seven parts of sand and stone. The mixing was done on a concrete floor. The dry materials were placed in layers in the following order, stone, sand and cement. The mass was then turned twice dry and then twice wet, the water being introduced from a hose. The mixture was then shoveled into barrows and wheeled to the molds and dumped over the sides; 4-in. to 6-in. layers were deposited the full length of the molds and tamped with iron tamping bars. The first two batches were moderately dry and the others pretty wet and required but little tamping. Very rigid knock-down molds were used and no special means employed to obtain a smooth surface on the concrete, other than that given by



*Vertical ⅛ inch corrugated bars placed as shown and not attached to horizontal bars.
**The ends of the reinforcing bars bent as shown in sketch below beam No. 11.

Diagrams Showing Construction and Dimensions of Concrete-Steel Beams Tested at Rose Polytechnic Institute.

mill finished lumber. The molds were not wet before being filled and were not removed until the beam was to be tested. The beams were seasoned in basement rooms of a temperature of about 65° F. The accompanying diagram and Table I. show the general dimensions of the beams and the arrangement of the steel bars. Although the beams were widely different in the dimensions of length and depth, yet all were designed on the same basis, the same percentage of steel being intended for each beam. From each batch of concrete two prisms, 6 × 6 × 9 ins., were made for compression tests. Unfortunately, however, so little care was taken in filling the molds that the specimens were very porous and non-homogeneous; consequently the results of the compression tests were of little value.

In testing a beam, the ends were supported in stirrups, free to swing on knife edges at about the center line of the beam. Two equal, concentrated loads were applied, symmetrically about the center of the beam, through knife edges in rolling seats, thereby producing a constant bending moment between the points of application of the loads (excepting the variable moment produced by the weight of the beam). The rolling seats for the knife edges were supported by ¾ × 4-in. steel plates, bedded in plaster of paris on top of the beam.

The vertical deflections were read directly from a scale on the side of the beam at the center by means of a silk thread fastened opposite the knife edges of the end stirrup. Measurements for the determination of the relative positions of neutral axis, under different loads, were made in the following manner: Four yokes were fastened to the beam by conical points. These were connected by wooden strips and the relative positions of these determined at the top and bottom by means of a silk cord fastened to one and running over a graduated arc fastened to the other. The first readings of the arcs were taken under a small initial load.

Table II. gives the result of the tests in terms of the loads, bending moments and mode of failure. Table III. was computed by Johnson's formula using the value of y_1 obtained from the experiments. This value was, in most cases, found graphically. For four beams, however, it was computed and the resulting values plotted. These lines showed that at the commencement of the loading the neutral axis was below the center of the beam. As the load increased in magnitude, the axis moved upward very rapidly until cracks commenced to appear on the bottom of the beam; then the axis remained approximately in the same position as long as the concrete did not show signs of failure in compression, as indicated by the drop of the scale beam. At or near the end of the experiment the axis, sometimes, not always, dropped suddenly.

The values of y_1 are probably in error, but not enough to be of any practical importance. The travel of the neutral axis is correctly obtained by the methods employed (assuming that a plane section remains plane after its distortion of the fibers), but its exact location in reference to the center of the beam is not known, as the experiments were commenced under an initial load, including the weight of the beam. The last two columns in Table III. were computed, using the same values of y_1, under the assumption that the concrete resisted no tension, and that in compression the distortion of the concrete was proportional to the stress producing it.

Throughout the entire series of experiments, the curves of deflection and extension showed a marked similarity.

*Abstract of a paper read at the April 6 meeting of the Western Society of Engineers.

†Professor of Civil Engineering, Rose Polytechnic Institute, Terre Haute, Ind.

TABLE I.—Showing Make-Up and Dimensions of Beams.

Beam.	Steel bar.	Nominal areas.	Actual net areas of steel.	Dist. Ctr. Steel below top of beam d.	Total depth of beam.	Breadth of beam.	Length of beam over all.
	No. In.	Sq. Ins.	Sq. Ins.	Ins.	Ins.	Ins.	Ft. Ins.
1	2 ⅜	0.36	0.36	4½	5	12	12 0
2	3 ⅜	0.54	0.54	6½	7	12	14 0
3	4 ⅜	0.72	0.72	8½	9½	12	17 0
4	6 ⅜	1.08	1.08	12¾	14	12	17 0
5	2 ⅝	0.74	0.60	8¾	10	12	13 0
6a	3 ⅝	1.11	0.90	13	14½	12	17 0
6b	3 ⅝	1.11	0.90	13	14½	12	17 0
6c	3 ⅝	1.11	0.90	13	14½	12	17 0
6d	3 ⅝	1.11	0.90	13	14½	12	17 0
6e	3 ⅝	1.11	0.90	13	14½	12	17 0
7a	4 ⅝	1.48	1.20	17¼	19	12	19 6
7b	4 ⅝	1.48	1.20	17¼	19	12	19 6
7c	4 ⅝	1.48	1.20	17¼	19	12	19 6
7d	4 ⅝	1.48	1.20	17¼	19	12	19 6
7e	4 ⅝	1.48	1.20	17¼	19	12	19 6
8	2 ⅞	1.10	1.06	12¾	14	12	17 0
9	3 ⅞	1.65	1.59	19¼	21	12	19 6
11	2 1	1.40	1.40	16¼	18	12	19 6

TABLE II.—Results of Tests Showing Maximum Loads and Moments.

Beam.	Age in days.	Span c-c End Support.	Dist. between Loading points.	Total weight of beam.	Total Max. Load.	Load when Comp. failure was marked. Load falling off from Max.	Max moment 1000s of in lbs.	Theoretical moment Johnson's Formula.
		Ft. In.	Ft. In.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.
1	74	10 0	3 4	825	4200	3400	94.3	80.6
2	76	12 0	4 0	1150	8100	5100	212.2	174.2
3	72	15 0	5 0	2000	12100	.....	402.7	322.2
[1] 4	73	15 0	0	2860	19400	.....	929.8	725.0
5	71	12 0	4 0	1700	11900	.....	311.9	283.0
6a	71	15 0	5 0	2935	18300	.....	607.2	625.0
6b	69	15 0	5 0	2950	19325	16500	638.3	625.0
6c	115	15 0	5 0	2450	20400	19100	666.3	625.0
[2] 6d	29	15 0	3 0	2960	15500	.....	616.8	625.0
[3] 6e	29	15 0	3 0	3125	17000	.....	692.0	625.0
7a	78	18 0	6 0	4450	30000	28500	1189.4	1121.0
7b	78	18 0	6 0	4525	29000	23000	1152.0	1121.0
7c	70	17 9	6 0	3800	30100	.....	1142.9	1121.0
[4] 7d	68	18 0	8 0	4450	29900	.....	1096.4	1121.0
[5] 7e	77	17 6	3 0	4200	22800	.....	1090.7	1121.0
8	77	15 0	5 0	2000	23300	22300	756.5	725.0
[6] 9	77	18 0	6 0	4850	34800	.....	1373.7	1652.6
11	75	18 0	6 0	4225	29000	25700	1149.3	1177.8

Notes.—[1]14,000 applied 6 ft. 6 ins.; removed and loaded in center; failed at load. [2]Failed by compression near center. [3]Failure same as 6d; made of gravel from pit. [4]Several repetitions of load; failed by shearing. [5]Repeated loads; failed by shearing. [6]Failed by shearing.

TABLE III.—Showing Fiber Stresses According to Johnson's Formula Experimental y_1.

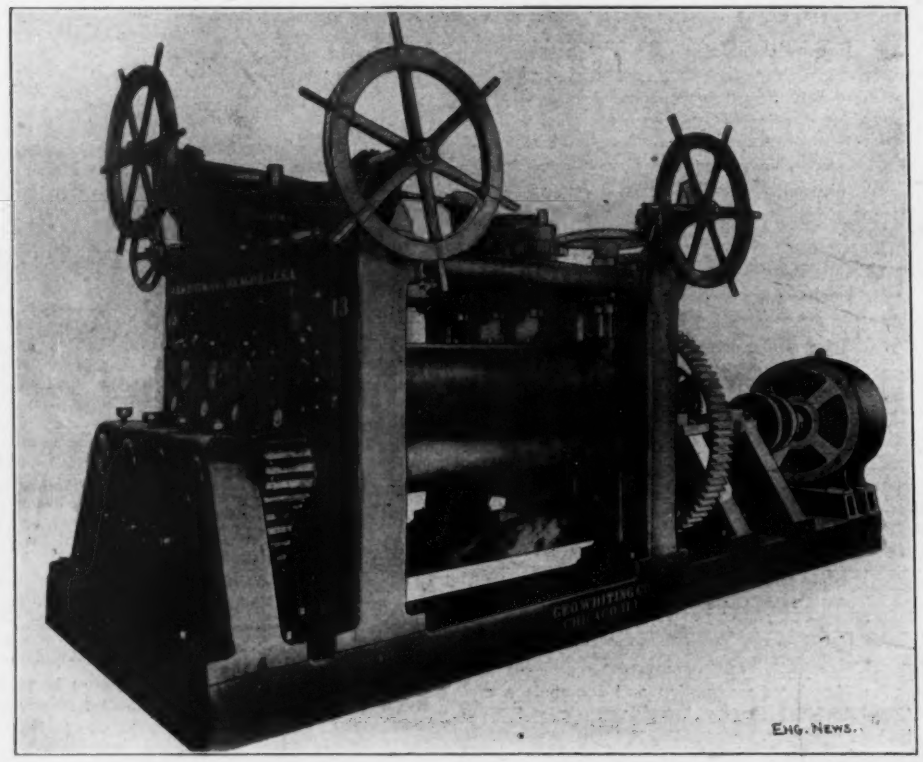
Beam	Centre of Steel Below Top of Beam. d	Neutral Axis Below Top of Beam. y	Max. Stress in Concrete in Comp. Lbs. per sq. in.	Concrete in Tension. Lbs. per sq. inch.	Steel in Tension. Lbs. per sq. inch.	Max. Comp. in Concrete. Lbs. per sq. inch	Tension in Steel. Lbs. per sq. inch.
	IN.						
1	4½	0.45d	1700	170	66000	2120	76500
2 *	6½	0.45d	1900	190	66000	2380	76500
3	8½	0.38d	2300	230	64000	2870	74300
4	12¾	0.34d	2500	250	64800	3100	75500
5	8¾	0.42d	1500	150	60500	1900	59000
6a	13	0.37d	1500	150	51000	1900	59300
6b	13	0.37d	1500	150	54000	1900	62500
6c	13	0.42d	1400	140	58300	1750	67700
6d	13	.....	.....	....	.....	.....	.....
6e	13	.....	.....	....	.....	.....	.....
7a	17¼	0.39d	1600	160	57000	2000	66400
7b	17¼	0.53d	1600	160	53500	2000	62000
7c	17¼	0.36d	1600	160	53500	2000	62000
7d	17¼	0.35d	1400	140	47000	1800	55500
7e	17¼	0.35d	1600	160	52200	2000	60000
8	12¾	0.36d	2000	200	54000	2500	64200
9	19¼	0.40d	1400	140	45000	1750	52200
11	16¼	0.39d	1700	170	50500	2100	58100

EIGHT-ROLL PLATE STRAIGHTENING MACHINE.

A large plate-straightening machine for handling plates up to 1 in. in thickness and 5 ft. in width has been built for the North Works of the Illinois Steel Co., by the Geo. Whiting Co., of Chicago, and a view of the machine is shown in the accompanying cut. The machine is of the eight-roll type, with all rolls 13 ins. diameter, and the two front rolls are used to "pene" or straighten plates edgeways. The front top roll is adjustable independently at either end and thus allows of pressure being brought to bear on either edge of the plate as it is passing through. The straightening rolls proper are mounted in the same housings, the top rolls being adjustable vertically, the back roll independently, and the intermediate top rolls together. Means are also provided for vertically adjusting all the top rolls together when once set.

Special attention has been given to means for

easily removing renewable parts without dismantling the whole machine. The lower roll bearings have removable bronze bushings throughout. The upper roll boxes are held in place by caps bolted on the outside of the housings and can be readily removed. The adjusting screws and nuts are arranged for removing without the necessity of disarranging any other parts, and all adjusting gears are readily removed in the same manner. The machine is mounted on a heavy bed plate 17 ft. 9 ins. long and 6 ft. 7 ins. wide over all. All the housings and gear-stands bridge across both sides, making a stiff and substantial construction. All driving gears are of cast steel, and all adjusting gears are cut from solid blanks. Hand wheels of large diameter are provided for adjusting the top straightening rolls and operating the pening rolls. Provision is also made for raising the top rolls a much greater distance above the bottom rolls than can usually be done. As will be seen in the cut, the machine is driven by a 25-HP.



EIGHT-ROLL PLATE STRAIGHTENING MACHINE.
The George Whiting Co., Chicago, Builders.

motor, having a flexible coupling between motor and gearing, thus relieving the motor from the shock of sudden overloading. The coupling consists of a short canvas belt running on a pulley formed by pegs between two disks, as shown. The machine can, of course, be driven by belt, steam engine or electric motor, as may be required.

SOME RECENT TESTS OF THE WEIGHT OF CROWDS.

By L. J. Johnson,* Assoc. M. Am. Soc. C. E.

The writer has for some time been coming to suspect that the accepted ideas as to the weight of a crowd of people (80 to 120 lbs. per sq. ft.) are somewhat wide of the mark. This suspicion was strengthened by a rough test some time ago when, with no extraordinary effort to crowd, twelve men were caused to stand in a floor space with vertical walls all round it 3 ft. 5 ins. × 4 ft., or 13.7 sq. ft. The aggregate weight of these twelve men was 1,839 lbs., averaging 153.2 per man, and yielding 134.2 lbs. per sq. ft. of floor.

This led to a series of three tests which give still higher results and which will now be described in detail as Tests A, B and C, in all of which the men were weighed one at a time just before entering the test area.

For Tests A and B, an alcove 4 ft. × 4 ft. 1¼ ins. was used. It was enclosed on three sides by vertical walls, and the fourth side was closed, after the alcove had been filled with men, by large drawing boards so as to prevent any projection beyond the actual limits of the alcove. The area within which the load was kept was 16.4 sq. ft. In these two tests many of the men wore overcoats or heavy overshoes or both, which accounts for the high average weight.

In Test A the alcove was made to contain 15 men, averaging 161.9 lbs. per man, aggregating 2,429.1 lbs., or 148.1 lbs. per sq. ft.

In Test B the corresponding figures for 15 men (two of whom only took part in the preceding test) were 157.4, 2,360.6 and 143.9 lbs. per sq. ft.

Next a larger area was tried in the belief that it might include a smaller proportion of waste space. This area was afforded by a rectangular room about 11 ft. by 6 ft., which, after adding the area of the door reveal and deducting the section of a rectangular ventilating duct fitting closely in one corner, offered with the door closed a net available area of 64.7 sq. ft., all measurements being taken above the baseboards. In this case none of the men wore overcoats. The result was as follows:

In Test C, 67 men entered the test room and the door was shut and latched. The average weight was 151.5 (the reduction being largely due of course to the absence of overcoats), the aggregate was 10,152 lbs., and the load per sq. ft. was 156.9 lbs.

The mean space per man was 0.97 sq. ft. in Test C, against 1.14 and 1.10 in the others. Such close packing as this was unexpected, and to test it further, the room was filled again two days later and the number of men required was, as before, 67, the crowds consisting mainly of the same individuals in the two cases.

It was found that with 33 men in this room, giving a load of 80 lbs. per sq. ft., the crowding was no greater than is common on our city sidewalks in the busy districts, and it offered no serious obstruction to walking through it. Eight more men entering and bringing the load up to 100 lbs. per sq. ft. visibly compacted the crowd, but it was still capable of motion, and one could make one's way through it, but with a little difficulty. Nine more men, bringing the weight up to 122 lbs. per sq. ft., compacted the crowd so that a man could elbow his way through it only with perseverance and determined effort. Seventeen more men still, making up the total of 67, turned the crowd into a decidedly compact mass of humanity, but there was no complaint of serious discomfort, but rather the contrary.

The men who took part in these tests were students taken wholly at random from the writer's classes. As students of engineering, and scientifically interested in the experiments, it was believed that they could be depended upon to pack themselves as skilfully as need be for practical purposes, and the economizing of space and crowding was left entirely to them. With 67 men in the room, there were calls from within for more men, and after making due allowance for the possibility that these calls were jocose in their nature, the writer believes that room might have been found by perseverance and severe crowding for another man at least, which would raise the total by some two or three pounds—an increase not important enough to justify the attempt to make it. If an extreme result were sought, the way to get it would be to select tall, slender men exclusively. Probably 160 lbs. per sq. ft. could be thus exceeded, but the result would be of little practical importance.

The 67 men included none of extraordinary height or stoutness. They are believed to be fairly representative of men of their age—about 20 to 22 years on the average. Five-sixths of them weighed less than 171 lbs., over half were under 151, about one-eighth under 131, and the extremes were 208 and 120.

In connection with this study, the writer has collected the following series of extracts showing the ideas in vogue in various countries. They form interesting reading in the light of the foregoing. It is curious that one only of the authorities cited (Stoney) gives a figure reasonably approximating the writer's results, and Stoney's data seem to have been overlooked and forgotten.

Whether the accepted figure as to the proper allowance (with the present unit stresses) be modified or not, the idea that 80 or 100, or even 120 lbs. per sq. ft., is the outside limit for the actual weight of a crowd should be allowed to prevail no longer.

Following are the quotations just mentioned:

On bridges for turnpikes and common roads, no probable contingency could crowd people to such an extent as to weigh more than 80 lbs. per sq. ft. of floor; and this may safely be taken as the maximum load on spans of 20 or more feet. To compensate, however, for impact, we recommend to adopt 100 lbs. as the limit for crowds. (This recommendation is believed by the writer to be generally accepted at present in this country. L. J. J.)—Trautwine's Pocketbook, 18th ed., p. 726.

Weight of men, women and children over 5 years of age, 105.5 lbs., and one-third of each will occupy an average area of 12 × 16 ins. = 192 sq. ins. = 78.5 lbs. per sq. ft. Of men alone 15 × 20 ins. = 300 sq. ins. = 48 in 100 sq. ft.

Bridges, Etc.—Weight of a body of men, as of infantry closely packed, = 138 lbs. each, and they will occupy an area 20 × 15 ins. = 300 sq. ins. = 66.24 lbs. per sq. ft. of floor of bridge, and as a live or walking load, 80 lbs. per sq. ft.

Weight of a dense and stationary crowd of men, 120 lbs. per sq. ft.—Haswell's Pocketbook, 67th ed., p. 841.

I packed 29 Irish artisans and 1 boy, taken from a forge and fitting shop, and weighing collectively 4,382 lbs., or 146 lbs. per individual, on a weigh-bridge 6 ft. 1 in. × 4 ft. 10 ins. = 29.4 sq. ft. In this experiment the men overhung the edges of the weigh-bridge to a slight extent and gave too high a result; I therefore on another occasion packed 58 Irish laborers, weighing 8,404 lbs., or 145 lbs. a man, in the empty deck-house of a ship, 9 ft. 6 ins. × 6 ft. = 57 sq. ft.; this gives a load of 147.4 lbs., or very nearly one man per square foot, and is, I believe, a perfectly reliable experiment. Such cramming, however, could scarcely occur in practice except in portions of a strongly-excited crowd, but I have no doubt that it does occasionally occur. The standard working load for suspension bridges in France was formerly 200 Kg. per sq. m., = 41 lbs. per sq. ft. This may be a sufficient standard for bridges with gatekeepers at the ends to prevent overcrowding, but it is obviously insufficient for bridges which are free to the public, especially in the vicinity of towns, and modern French practice seems to have raised the standard to 82 lbs. per sq. ft. Drewry adopted 70 lbs. per sq. ft. of platform as the greatest load that a public bridge would sustain if covered with people. Tredgold and Professor Rankine estimate the weight of a dense crowd at 120 lbs. per sq. ft., and the late Mr. Brunel is said to have used 100 lbs. in his calculations for Hungerford Suspension Bridge. Mr. Hawkshaw adopted 80 lbs. per sq. ft. for the footpaths of Charing Cross Bridge; and (in conjunction with Mr. W. H. Barlow) 70 lbs. for the Clifton Suspension Bridge. My present opinion is that the standard working load of public bridges for calculation should not be less than 100 lbs. per sq. ft. of platform, especially for bridges near cities.—Stoney, Theory of Strains, II., p. 400 (Ed. of 1869).

While 80 lbs. is the maximum possible live load per square foot from a crowd of people (unless dancing is considered), still we can hardly expect to realize any such load under the conditions governing an office building.—Freitag, Architectural Engineering, 2d ed., p. 115.

Only in cases where people are liable to be jammed together during a panic or some unusual circumstance is

*Assistant Professor of Civil Engineering, Harvard University.

it possible to get a weight on the floor of 120 lbs. per sq. ft.—Kidder's Pocketbook, 13th ed., p. 426.

Weight of a closely packed crowd estimated at 120 lbs. per sq. ft. (for timber bridges and platforms).—Rankine's Civil Engineering, p. 466.

Weight of a crowd of people should be taken on the average (5 or 6 men to the square meter) at 400 kilograms per square meter (82 lbs. per sq. ft.). For country bridges 300 (61 lbs. per sq. ft.) might be a sufficient allowance; for city bridges 500 kilograms per square meter (102.4 lbs. per sq. ft.) would be preferable.—Trans. from Hütte Taschenbuch, 18th ed., II., p. 225.

For a very compact crowd, like a column of soldiers, occupying the whole length of a bridge, the weight per square meter is less than 200 kilograms (41 lbs. per sq. ft.). For a crowd so compact as to be able to move only with difficulty, the weight may rise to 320 kilograms (65.6 lbs. per sq. ft.). Finally, if the crowd is so compact that no motion is possible, and accidents are imminent, this weight may attain the maximum of 400 kilograms (82 lbs. per sq. ft.), which supposes 7 or 8 persons per square meter of the roadway.—Trans. from Résal, Ponts Métalliques, p. 70.

450 kilograms per square meter (92.2 lbs. per sq. ft.) should be taken as the weight of a crowd of people not too compact for slow motion. For such portions of bridges as are likely to have to carry a more compact crowd, such as sidewalk cantilevers and the like, 500 kilograms per square meter (102.4 lbs. per sq. ft.) should be assumed. —Free trans. from Landsberg, in Handbuch der Ingenieur Wissenschaften, II., 2, pp. 24, 25.

CURRENT PRACTICE IN APARTMENT-HOUSE CONSTRUCTION IN NEW YORK CITY.

In connection with the discussion on our editorial page of some phases of apartment-house construction in New York city, we present herewith a photograph of a typical building of this class in course of erection, which illustrates well a number of the objectionable features of present practice. The structure is 11 stories in height and covers an area about 80 × 100 ft. in plan; it is of iron frame and "curtain-wall" construction, with floors of reinforced concrete. At the time of the photograph the iron framework was nearly completed, while the walls had only just been started. Three or four stories lie below the limit of the picture. The iron work shown is very light throughout, and is, moreover, of peculiar type. The columns of the lower half are of cast-iron, while those of the upper half are of steel. The change is made for the reason that steel sections make the cheaper column after the limiting minimum size of cast-iron column fixed by the Building Law is reached. In this building the change was made at about the middle of the height. The columns of the upper half are made of I-beams (channels?), double angles, or single angles. This type of construction, though uncommon, is not wholly new, but has been used in New York city in half a dozen or more structures of similar character built during the past few years.

The framework shown in the photograph has no provision for permanent bracing, and the connections between beams and columns have practically zero rigidity. The details at the walls are particularly bad, although no worse than may be found in many buildings of this class. The columns being set within the walls, while the floor-beams are clear of the walls, eccentric connections on all wall columns result. At the corners this is particularly noticeable, since here the eccentricity in both directions has a resultant nearly one-half greater than at other wall-columns. Where the corner column is of cast-iron, the detail at this point is a diagonally-projecting bracket (without lug for web-connection) whose eccentricity of loading is about 12 ins.—with a column only 6 ins. in diameter. Where the columns change to steel, the details are little, if any, better. The upper half of this building, in fact, is of precisely that class of steel construction which, in our editorial in this issue (and also in the discussion of the Darlington case in our issue of March 24) we have characterized as being not a particle better than cast-column construction.

FRAMEWORK OF UPPER STORIES OF A NEW YORK APARTMENT HOUSE NOW UNDER CONSTRUCTION.

SOME COMMENTS ON BUILDING LAWS.

By Gunvald Aus, M. Am. Soc. C. E.*

Two months ago the writer opened a discussion in the Brooklyn Chapter of the American Institute of Architects by reading a paper on "Some Engineering Points in the New York Building Code." The questions raised seemed to be of great interest to members present as well as to the Chief Engineers of the Bureaus of Buildings of the Boroughs of Manhattan and Brooklyn, both of whom took part in the discussion. The following contains the writer's remarks, somewhat elaborated:

When an engineer is called upon to design a bridge structure or the supporting elements of a building, he naturally begins his work by inquiring what loads are to be sustained and to what extent he will strain the structural elements. The loads are generally of four distinct varieties: (1) the weight of the structure; (2) the load to be supported on the floors; (3) the wind pressure; and (4) the weight of derrick while raising the heaviest element of the structure.

The first of these items is easily ascertained, although one is apt to get too small weight by accepting the statements of the manufacturers of different floor constructions.

*Consulting Engineer, 79 Wall St., New York City.

FLOOR LOADS.—This second item is very much more difficult to establish. For bridges we have a number of admirable specifications, as, for instance, Cooper's Specifications, which give the load for different classes of bridges; the question of which class to select depending on local conditions and the pocketbook of the owner. For buildings, especially such as are to be occupied for offices, dwellings or assembly rooms of different kinds, it is on the contrary very difficult to arrive at a solution satisfactory to all parties concerned.

Let us, for instance, take the case of a private dwelling, as ordinarily built in New York. Such a house generally consists of a basement and two or three stories, covering an area of some 20 × 50 ft. We know that a well-furnished house of this character contains two or three loads of furniture weighing probably from two to five tons, and that the heaviest piece of furniture, the piano, will weigh 800 to 1,200 lbs. There will, in all probability, never be more than one ton in any one room, except in specially designed large libraries where the question of sustaining book cases can be treated as a separate proposition. For the average room of some 14 × 18 ft., the load will probably not exceed 2,000 lbs., or say 8 lbs. per sq. ft. It would seem then that if we assumed a load of 10 lbs. per sq. ft. and a possible concentration of 1,200 lbs. over an area of about 3 × 6 ft., the building would be perfectly safe as far as this load is concerned. A further consideration that would have to be dealt with in this case is the deflection of the floor joints, which, of course, must not be so great as to crack the plaster. But what does the building law say about this? That any part of these floors must be able to sustain a superimposed load of not less than 60 lbs. per sq. ft. In other words, every floor must be so strong that it can be used for storage of furniture, not excluding pianos, over the entire floor.

In case of an office building, the difference between real loads and those assumed by the Building Code is as great; and even then the present Code does not insure a perfectly safe floor. We all know that near elevator shafts and large vents convenient recesses are formed where heavy safes are often placed. But in such recesses the spans are so short that only very light beams are called for, so light in fact that they are often overloaded by a heavy safe, while the entire balance of the floor is not loaded more than probably 10 to 20 lbs. per sq. ft. Would it not therefore be much more rational to specify a smaller uniform superimposed load and a concentration of perhaps one ton on an area of probably 25 sq. ft? If it is desired to use heavier safes, a special permit could be obtained and the floor, if necessary, reinforced.

But worst of all appears to me the provision that the first floor shall be able to sustain 150 lbs. superimposed load; in other words, shall be as strong as the floor for a heavy warehouse and very much stronger than the floor of the average highway bridge. Is it made so heavy because building materials are stored here? Probably. But would it not be an easy matter to shore up this floor during construction; and have we not all seen that such building materials are as often piled on the upper floors?

For assembly rooms the Building Code specifies 90 lbs. per sq. ft. superimposed loads, and for buildings for light manufacturing purposes 120 lbs. While it may be possible to crowd people to such an extent that a small section of floor would carry 60 to 80 lbs. per sq. ft. once or twice during its life, it is also well known that ordinarily assembly rooms are not loaded over 20 to 40 lbs. per sq. ft., and most floors occupied by light machinery are not loaded over 20 to 50 lbs.

Just how much the 90 lbs. load specified for assembly rooms could be reduced is, of course, to a great extent a matter of opinion, but it would seem that 60 lbs. per sq. ft. would be entirely safe. As the weight of the floor will average 70 lbs. per sq. ft., the total load will be 130 lbs. per sq. ft., or during some particular crush, possibly 150 lbs., causing an excess of stress that would certainly not be serious as it would only possibly occur a few times in the life of a floor.

For factories of all classes, and warehouses, it

would seem more rational to group these according to the actual loading, allowing probably 60 lbs. per sq. ft. for the lightest class and increasing to 600 lbs. or more for the heaviest—the safe load to be posted on each floor, and inspection made to see that this load is not exceeded, in which case a heavy penalty should be incurred and enforced.

At whatever load, however, one agrees to proportion the floor joist, a considerable reduction should be made for the girders and columns and a still greater for the foundation. Any one who has had opportunity to look into the loading of office buildings, factories and even storage warehouses, will know that only a few joists carry the full load so that the girders and columns only carry a fraction of the full load. By actual measurements in several warehouses occupied by the United States as "Appraisers' Stores," I found the worst condition to be two-thirds of the floor area carried by any one girder fully loaded. It would therefore seem proper to proportion the girders for the full dead load and two-thirds of the superimposed load for which the floor joists are proportioned. The upper two tiers of columns should be proportioned for the same load as the girders, and a reduction of probably 10% might be made at each story, until a reduction of say 30% of the superimposed girder load is reached.

The foundations should be designed for the full dead load, and not over 60% of the live load assumed for the basement columns. Where the ground is yielding this latter reduction is very essential, as otherwise piers which carry a considerable portion of the wall and only a small floor area will at once receive nearly the full load for which they are proportioned, while interior piers carrying only floor load will receive only a fraction of the load for which they are proportioned, causing unequal settlement with resulting cracking of walls and plaster. I had opportunity some years ago to notice a case of this kind in Omaha, Neb., where a massive Government building is erected on a clay foundation. The piers were all proportioned for the full live load, with the result that the foundations under the walls settled very much more than the interior foundations, and the corners, which supported only a very small floor area, settled much more than the balance of the wall.

In certain classes of warehouses it does, of course, happen that the entire floor area is loaded, and in such cases no reduction for girders, columns or foundations would be permissible. In such cases the foundations should in no case be laid on a yielding strata, but should invariably be carried to gravel, hard sand or rock or be piled where no good bottom can be found.

WIND BRACING.—It has been a much disputed question how much wind bracing is necessary. The Building Code specifies an assumed wind pressure of 30 lbs. per sq. ft., which probably is ample over the very large area of a building. But when permission is granted to omit all bracing in buildings of only 10 to 12 stories, reliance being placed on the bracing given by partitions and walls. However, the partitions are often omitted permanently on the first floor and in many cases are not put in on the upper floors until the building is accepted or at least rented, when the tenants' wishes as to subdivision can be learned. Especially where cast-iron columns with loosely bolted connections are used, the building has therefore practically no resistance to horizontal stresses during erection and long after completion.

Would it not seem reasonable to insist that sufficient wind bracing shall in all cases be provided to take care of a wind load of 10 lbs. per sq. ft., entirely regardless of the stiffening of partitions and walls? It is, of course, in the owner's and designer's power to put in all the bracing needed, but as the tall buildings* erected to-day are of a speculative nature, it is generally not possible to provide anything better than the minimum that will pass the Building Department.

ERECTION STRESSES.—Before discussing the permissible stresses in the materials of construction, I would like to say a word of the erection stresses, which are not generally considered.

*Office buildings should be excepted from this statement.—Ed.

The stiff leg derrick now almost universally used in erecting structural materials for buildings, exerts a pressure under the center casting of sometimes more than twice the load lifted plus the weight of the derrick, and yet you will find that generally no precaution is taken to take care of this load, which may be several times as great as that for which the beams on which the derrick rests are proportioned. Moreover, such a derrick exerts severe horizontal stresses, and these are more or less jerky, depending a good deal upon the manipulation of the derrick.

As to-day most buildings of ten to twelve stories have no resistance to horizontal stresses until the walls and partitions are finished, it is probable that the joints of beams and columns are badly strained. It would hardly be justifiable to proportion the floor beams to sustain the load of the derricks, as simple shoring under the center part can easily be put in, but as at present no wind bracing is generally provided in such buildings, I believe a certain amount of kneebracing should be inserted as a permanent part of the steel frame. The ordinary chain and turnbuckle will do the work, if properly put in; but they are often not put in place until after the building is found to be out of plumb, and then often drawn up so hard as to pull the building out of plumb the other way, causing a loosening of all joints when the chains are finally taken away.

MATERIALS AND SAFE STRESSES.—As structural material, steel and—I am sorry to say—cast-iron are the only two elements used.

The safe loads on steel beams and columns are fairly well established, and the stresses allowed by the Building Code are very safe and conservative, although I am sure a majority of engineers would be willing to allow more than 16,000 lbs. per sq. in. on rolled beams, where the load to be sustained is all practically a static load as in most buildings, not used for manufacturing purposes.

Very different, however, is the case where cast-iron columns are concerned. For an average story height, the Building Code allows about 12,300 lbs. per sq. in. for a steel column, and 9,800 lbs. per sq. in. for a cast-iron column; in other words, the steel column is credited with only about 25% more strength than the cast-iron column. This is probably approximately a proper ratio where both columns are very short and both perfect and the load absolutely concentric. But in practice the load is not concentric and considerable bending stresses exist in all columns. Furthermore, it is impossible to inspect the cast-iron columns, as only the outer face can be examined. Drilling a few holes to ascertain the thickness is of some use, but does not in the least help to show whether the column is full of cinders or blow holes. Anyone who inspects such columns will often find an apparently sound column to be in spots nothing more than a thin shell of iron covering a body of cinders.

In one Government building where the cast-iron columns had been inspected and accepted, I accidentally discovered several such spots in a number of 12-in. and 16-in. columns, and with a light hammer broke in holes 6 to 8 ins. in diameter. The floors were shored up and new columns put in, in this case. But in how many buildings with 6 to 8-in. columns must we not suppose that a large part of the section is not to be relied upon, and how large do you suppose the factor of safety is in such a column?

Structural engineers will remember an accident that occurred some years ago in the Ireland Building in New York City. It has, of course, been claimed—and probably truthfully so—that the accident was not due to any defects in the cast-iron columns, but entirely due to a failure of a large and thin foundation stone. However this may be, the fact remains that owing to something giving way, either the stone or the columns in the basement, a number of columns broke and a large section of the floor fell in; an accident that could never have occurred with steel columns under the same conditions. At that time, Mr. Stevenson Constable was Superintendent of Buildings in New York City, and he caused a number of columns to be tested to destruction on the large testing machine owned by the Phoenix Iron Co. at Phoenixville, Pa. The writer was at the time Chief Engineer in the Supervising Architect's office in charge of Public Buildings, and was present at the testing in order to watch the result, as there was a keen competition at the time in Washington between cast-iron and steel columns. I have not now the data at hand, but am quite sure that these cast-iron columns, although they appeared to be an unusually nice lot, failed at anywhere from one-third to one-half the ultimate strength with which they were credited according to the Gordon formula; in some cases, I believe, they failed at even less load, and this although the loading was made in the most careful manner, as nearly concentric as it is possible to produce by artificial means; and certainly very much more nearly concentric than it ever is in practice. I therefore feel convinced that the factor of safety of 7, which would seem to be that required by the New York Building Code, in reality does not exceed 2, and I am very much afraid that it is sometimes very much less. In fact, I firmly believe that there are buildings to-day having cast-iron columns which do not fail for the simple reason that the columns never receive anything like one-half the load for which they are calculated.

Does it not then appear unreasonable to use cast-iron columns with any such stresses as those permitted by the building law, especially in view of the extremely weak connections between the columns of the different stories and between the columns and the beams and girders supported by them?

By comparing a typical cast-iron column as proportioned in accordance with the Building Code and the tables given in the Carnegie Pocket Book, you will see that where the former allows about 9,300 lbs. per sq. in., the latter allows only about 5,100 lbs.; yet I am sure most engineers who have had experience in inspection will feel much safer with a steel column loaded to 12,300 lbs. per sq. in. than with the cast-iron column at 5,100 lbs., which is little more than one-half of the load allowed by the Building Code.

Let us consider the thickness of the walls specified by the New York Building Code. The walls which carry no load except their own weight are of two kinds: those in a skeleton building, which are supported at each floor, or sometimes at every alternate floor and the "curtain walls," so-called, which are walls which carry their own weight from the foundation up to the top of the coping without supporting any floor. Now, the code specifies that the walls in a skeleton building, where they are supported at each floor, shall be 12 ins. thick for the upper 75 ft., and thence downward 4 ins. thicker for each 60 ft.; curtain walls, which carry themselves, shall be 12 ins. thick for the upper 60 ft. and thence downward 4 ins. thicker for each 60 ft. Thus there is very little difference in the requirement whether the wall is supported at every floor or carries its own weight for the entire height of the building.

This seems rather unreasonable, as the extra increase of thickness for the walls in the skeleton building cannot be of any other possible use than to load the building down, and this is certainly very unnecessary, unless the building should be so narrow in comparison with its height that it becomes necessary to introduce this extra weight to prevent the overturning of the building in a heavy wind. I have often wondered why this provision was inserted in the building law, and have come to the conclusion that it must have been done because the cast-iron columns, owing to the very poor connection between the different stories, and the very small resistance of the cast iron to bending, could not very well resist the wind forces to which the building is exposed, and so this great extra weight was introduced to help out the poor cast-iron columns. I do not, of course, know whether this was the reason for introducing this provision, but if it was, I think it is very sound reasoning. But I believe the building law should call for this increased thickness only in cases where cast-iron columns are used, and should permit the use of 12-in. walls (for walls carried by the steel framework) no matter how high the building, in cases where the columns are of steel and the connections are riveted, in other words, where the proper resistance to the wind pressure has been given.

In conclusion permit me to suggest how, in my opinion, good and safe buildings should be secured; not that I consider my opinion of particularly great value, but in the hope that I may give some suggestions which may cause discussion that will lead to a practical solution of the problem.

In the first place, establish a very much smaller floor load than now called for, and a suitable concentration to provide against local heavy loadings. Allow a suitable reduction for girders, columns and foundations.

Then, do not permit the use of cast-iron columns in buildings over four or five stories high; further, reduce the permissible stresses in such cast-iron columns and allow no such columns having a ratio of length to diameter of more than 18, so that nothing smaller than an 8-in. column can be used in a 12-ft. story.

Make the frame strong enough to resist 10 lbs. wind pressure regardless of the stiffening effect of partitions and walls, except in the case of buildings where the walls carry the beams, and allow the walls in skeleton buildings to be only 12 ins. thick, no matter how large the building.

Above all, insist on inspection at the mill, in the shop or yard, and at the building.

Why not have the Building Department license engineers to do this inspection and insist that a certificate be filed, signed by such engineers, stating that every piece of steel and iron has been inspected and that they are all sound, and of the size called for by the approved plans. Let the Department keep its own inspector as a check on the inspector employed by the owner, but do not attempt to shift the responsibility over on the shoulders of a building superintendent who is only in his position for a short time, and whose action may be influenced by political exigencies.

REINFORCED CONCRETE ROOF FOR A LOCOMOTIVE ROUNDHOUSE, LONG ISLAND R. R.

The problem of securing a locomotive roundhouse roof which will be free from corrosion and at the same time fire-resisting, has been solved recently by the use of a reinforced concrete roof construction carried on the usual brick walls and iron columns or columns of concrete. One of the first railways to use this construction was, we believe, the Canadian Pacific. More recently Mr. H. A. Mallery, Engineer of Bridges, Long Island R. R., has employed reinforced concrete in replacing a steel roof on the Morris Park roundhouse of the Long Island R. R. This house is of the usual ring shape, and embraces 180° of the circle, the inner diameter of the ring being 88 ft. and the outer diameter 160 ft. 4 ins. The original roof of this building was of steel, and it had become practically destroyed by the locomotive gases. In planning to rebuild, a wooden roof was first considered, but this construction was discarded, because of its danger of taking fire, and a concrete-steel construction was substituted. This construction is shown by the accompanying drawings.

The Morris Park roundhouse had a brick wall on the outer perimeter and cast-iron columns spaced 12 ft. apart on the inner perimeter. These were left in place, but the interior columns and the roof of the original structure were replaced by new construction. The drawings of Fig. 1 show a plan of one bay of the roof and a section taken radially through this bay. A circular girder of reinforced concrete was set on the cast-iron columns, and from each column point to the opposite brick wall a radial girder was built with two intermediate reinforced concrete column supports. This girder system supports and was built in one piece with a reinforced concrete slab roof pierced by an annular ventilator opening, as is shown by the drawings. The edges of this ventilator opening were stiffened by a girder-like thickening of the roof slab. This thickening was also desirable to give a stiff connection for the wooden uprights of the ventilator.

The girder and slab construction and their reinforcement are fully shown by the drawings of Fig. 1. As will be seen, they are very simple. Altogether 83,582 lbs. of reinforcing rods were used in the roof and columns. The only details that call for more particular mention are those shown in Fig. 2, and these are sufficiently explained by the drawings. The expansion joints are situated over the radial girders and divide the roof into groups of two and three bays. The materials and methods of construction were specified by Mr. Mallery to meet the following requirements:

FORMS.—The contractor shall build the forms from matched and dressed siding, held in place in such a manner that it will not be deflected by the ramming or the weight of the concrete. The faces against which the concrete is to be placed shall be dressed so that when the concrete is laid, it will present a smooth surface which will conform to the lines shown in the drawing.

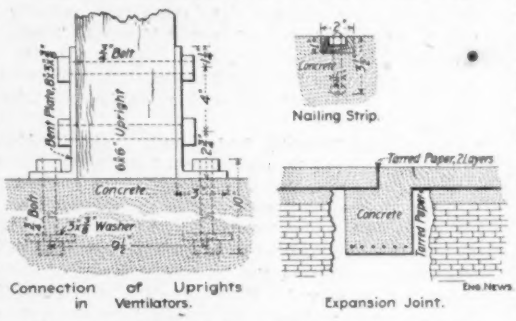


Fig. 2. Special Details of Concrete-Steel Roof for Long Island R. R. Roundhouse.

CONCRETE.—Concrete in piers shall be composed of 1 part cement, 3 parts sand and 5 parts stone. All other concrete used in the work shall be composed of 1 part cement, 2 parts sand and 4 parts broken stone. The sand shall be clean, coarse and sharp. If it is found to contain more than 4% of loam or silt, or material that is friable or soluble, the contractor will be required to wash the sand, before it is brought to the work. The stone used shall not exceed 1 in. in diameter, except in piers where it can be 2½ ins. in diameter. The crusher shall be set to produce the required size and the smaller product of the crusher can be used except that the dust shall be screened out.

MIXING.—The contractor may use a machine for mixing. The machine used shall be of an approved kind for which the proportions of each batch are exactly measured, and which will thoroughly mix the concrete, so that every stone is completely covered with mortar.

If the concrete is mixed by hand, the following method shall be used: One-half of the sand to be used shall be spread evenly over mortar box or platform. The cement shall then be spread evenly over the sand and the remainder of the sand spread on top. The sand and cement shall then be thoroughly mixed by turning and returning at least six times. This shall be done in such a manner that the number of times the material is turned can be counted by the inspector. The mixture shall then be drawn down to one end of the box and water poured in at the other end. The mixture shall then be drawn down into the water in small quantities as determined by the inspector, and mixed vigorously until there is a good stiff mortar. The stone shall be thoroughly drenched with water while in the pile. The measured quantity for each batch shall then be spread on a plank bed in a layer not more than 4 ins. thick. The measured quantity of mortar shall then be spread evenly over the layer of stone and the mass shall be mixed by turning and returning it three or more times, at all times preserving the thickness of the mass, until every stone is completely covered with mortar.

PLACING THE STEEL RODS.—Upon the bottom of the forms of the girders and slabs, a layer of mortar shall be placed sufficiently thick to support the rods at the specified distance from the bottom of the concrete. On this layer the rods shall be placed and stone shall be placed under the rods in such a manner that the ramming will not drive the rods through the mortar. To each batch of concrete used in the layer over the rods shall be added a quantity of stone equal in bulk to the mortar under the rods. The rods used will be the Thacher patent rods for concrete-steel.

LAYING.—After the concrete is mixed it shall be laid immediately in sections, as follows: In the base, the concrete shall be laid in 6-in. layers and allowed to set six hours. Each column must be laid in one day, from the bottom to the point where it joins the 24-in. girders, but it shall be rammed in 6-in. layers. Each girder shall be laid and rammed in 6-in. layers and shall be finished between centers of supports in one day. Each slab forming the roof shall be laid in one layer and finished in one day. No concrete over 45 minutes old shall be used in the work. All concrete shall be laid in sections parallel to the steel rods. In each layer the batches shall follow each other so closely that each one shall be placed and rammed before the preceding one has set. After setting, the concrete shall not be walked on in 12 hours.

RAMMING.—After each batch is laid, it shall be rammed with two-man rammers, so as to compact the concrete perfectly and fill all voids and shall be continued until the water flushes to the surface.

JOINING NEW CONCRETE TO CONCRETE THAT HAS SET.—When new concrete is to be joined to concrete that has set, the old work shall be thoroughly

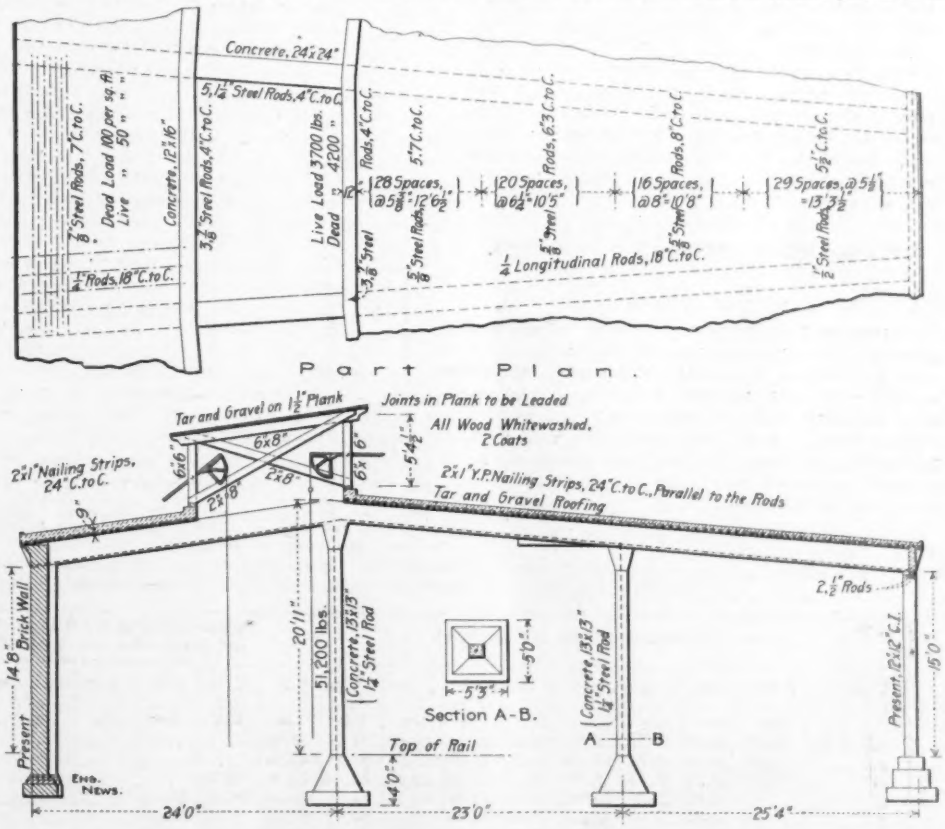

Concrete, 24"x24"
5, 1 1/4" Steel Rods, 4" C. to C.
Dead Load 180 per sq. ft.
Live " 50 " "
Concrete, 12"x16"
7/8" Steel Rods, 7" C. to C.
1" Rods, 18" C. to C.
3 1/2" Steel Rods, 4" C. to C.
Live Load 3700 lbs.
Dead " 4200 "
3/4" Steel Rods, 4" C. to C.
5/8" Steel Rods, 5'7" C. to C.
5/8" Steel Rods, 6'3" C. to C.
5/8" Steel Rods, 8' C. to C.
1/2" Steel Rods, 5 1/2" C. to C.
28 Spaces, @5 5/8" = 12'6 1/2"
20 Spaces, @6 1/2" = 10'5"
16 Spaces, @8" = 10'8"
29 Spaces, @5 1/2" = 13'3 1/2"
1/4 Longitudinal Rods, 18" C. to C.
Part Plan.
Tar and Gravel on 1 1/2" Plank
Joints in Plank to be Leaded
All Wood Whitewashed, 2 Coats
2"x1" Nailing Strips, 24" C. to C.
6x8"
6x6"
2x8"
2"x1" Y.P. Nailing Strips, 24" C. to C., Parallel to the Rods
Tar and Gravel Roofing
9"
14'8" Brick Wall
Present
20 1/2"
51,200 lbs.
Concrete, 13x13" 1 1/4" Steel Rod
2 1/2" Rods
Present, 12'x12" C.I.
15'0"
5'0"
5'3"
Section A-B.
Top of Rail
A
B
4'0"
End. News.
24'0"
23'0"
25'4"
Cross Section.

FIG. 1. PLAN AND SECTION OF ONE BAY OF CONCRETE-STEEL ROOF FOR MORRIS PARK ROUNDHOUSE, LONG ISLAND R. R.
H. A. Mallery, Bridge Engineer, Long Island R. R.

cleaned and washed and a thin layer of mortar, composed of equal parts of cement and sand, shall be spread over its surface. A groove shall be left in the top of the girders when the concrete in them has set before the roof slab is built, so as to form a bond between the slab and the girder.

EXPANSION JOINTS.—Where called for on the plan, expansion joints of tarred paper shall be placed to allow the slab to slide on the girder.

REMOVING FORMS AND FINISHING.—The forms shall be left in place for 28 days. When removed, any open or porous places shall be pointed up with mortar composed of equal parts of sand and cement. The faces of the work shall then be washed with neat Portland cement, after being thoroughly wet with water.

SPRINKLING.—All exposed surfaces shall be sprinkled with water once a day for 10 days after completion and shall be kept covered with canvas until thoroughly set.

We are indebted to Mr. H. A. Mallory for the information from which this description has been prepared.

A USEFUL TABLE OF WEIGHTS OF CAST-IRON PIPE AND FLANGES.

We reproduce herewith a table giving the thickness and weights per lineal foot of cast-iron pipe from 3 to 48 ins. diameter. The table has been compiled by Mr. Carlisle Mason, Chief Engineer of the Clinton Foundry & Machine Co., with offices at 95 Liberty St., New York City, and is published by that company.

The table has been computed on the basis that 1 cu. in. of cast iron weighs 0.2607 lb. The standand weight of pipe shown for 100 lbs. pressure is not theoretically correct in every case, but is close enough for practical purposes, the thickness being that commonly used for this pressure, and the factor of safety being large enough to allow of any small variation. The flanges shown on the right-hand side of the table are the American Society of Mechanical Engineers' standard.

While the table was compiled with especial reference to flanged pipe, it can also be used for cast-iron water pipe by taking the figures per ft. for pipe of the given diameter and thickness, multiplying by the number of feet in each length of pipe exclusive of the hub and spigot and adding the weight of the hub and spigot to obtain the total weight per length.

ELECTRIC CONDUIT CONSTRUCTION AT MEMPHIS, TENN.

By F. G. Proutt.*

Underground conduits for electric wires are coming into such general use in our larger cities that information concerning the engineering of conduit work and the cost of installing conduits must needs be useful to owners and managers of electric lighting and other companies who may be called upon to remove poles from the streets in certain portions of their cities or towns and place the wires underground.

*Electrical Engineer, Memphis Consolidated Gas & Electric Co., Memphis, Tenn.

The preliminary engineering work includes the planning of the conduit system, with the street location of ducts and manholes; deciding on style and number of ducts to be used and method of laying, locating manholes and service boxes, preparing the drawings, estimating the cost of the work, and getting matters in such shape that bids may be asked or the work carried out by day labor under the supervision and general direction of the company's engineer. As the writer recently put in about 250,000 ft. of duct at Memphis, Tenn., first planning the system, then making the estimates, and finally having all the work done by the day, this article will deal with work done in this way.†

The city of Memphis had already a district 1½ × ¾ miles served by underground conduits. The plant of the Memphis Consolidated Gas & Electric Co. is situated ¾-mile east of the eastern limit of this underground district, and it was decided to install a duct system to connect the plant with that district, so that the business portion of the city would be served with electric light and power by a complete underground system from the generating plant to the customers' premises. For this purpose there were required when the work began, 12 ducts for 2,300-volt alternating current feeders, 4 ducts for 500-volt direct-current feeders, 2 ducts for three-phase alternating current feeders and 3 ducts on each run for distribution purposes, making 21 ducts if all were run on one street, or 24 ducts if runs were made on two streets.

The plant is situated on Beale St., some distance east from the south end of the old underground district. If all ducts were run down Beale St., it would have been necessary, on reaching the underground district at Hernando St., to run a number of cables to the north end of the district and put in enough extra ducts for future business. In time the question of distributing the cables from Beale and Hernando Sts. to their various terminals would have been a serious one, consequently it was decided to make two runs from the plant, the plant then being at one point of a triangle, as shown in Fig. 1; and then branching one of the feeders so that three sets of feeders are connected with the underground district about equally distant apart. This method requires 24 ducts for present requirements.

As the city is growing very rapidly it was estimated that at some future time the load would be double that at present, but if it increased more than this it would be better to put in a duct line on another street. It was therefore, decided to put 27 ducts on Beale St. to De Soto St. and branch at this point, continuing 18 on Beale St. to Hernando St., and running 18 to Monroe St. The two 18-duct runs make 36 in place of 27, but the object of putting 18 in each case in place of 12 and 15 was that the cost was very little more and it gave a choice of taking 18 cables on either run, which might be a great consideration at some future time should heavy business develop in some particular locality. On the Alabama St. run it was decided to put 18 ducts, making a total of 45 ducts from the plant, with present requirements of 24. Deciding on how many extra ducts to put in is a matter of personal opinion and must be determined largely by the amount of money that the electric company cares to spend.

†The work at Memphis was done by day labor. Similar work done by contract at Cincinnati, O., was described in our issue of Aug. 20, 1903.—Ed.

After deciding on the number of ducts and the streets on which they shall be run, comes the location of manholes and service boxes. As a general rule manholes should be placed at all street intersections and the engineer should, with two or three assistants, walk over the proposed duct route. One of these assistants should carry a bucket of black asphalt paint and the other two a tape line. Beginning at one end of the line, a manhole should be located and a spot painted on an adjacent wall or fence to mark its center. For convenience in distinguishing manholes from service boxes, a cross for the former and a circle for the latter answer very well.

The men with tape line then start from the first manhole mark, and the engineer should have a city map of some kind. On this he locates the manholes, then the service boxes between the manholes, and marks the distances, so that when a section of street has been gone over a rough map has been made showing locations of manholes and service boxes and distances between them. When construction begins the paint marks give a ready means of finding locations.

Manholes should not be over 500 ft. apart, unless in exceptional cases; but may be 500 ft. without giving trouble in pulling the cables, provided, of course, that the run between manholes is reasonably straight. The service boxes should be located entirely with reference to buildings along the route and should be, as a rule, located so that the center of the box is on a line between two lots on one side of the street or the other. In other words, all boxes should be so located that the laterals from them may be run to the greatest number of buildings with the shortest laterals possible. Some boxes may be only 80 ft. apart and some may be 220 ft., but they will average about 150 ft. As already stated, the ducts were put in for running feeders to the old underground district, but in all underground work the distribution system along the route should be taken care of while the work is being done and this is the purpose of the service box. A manhole and service box are shown in Fig. 2. The service boxes, as shown, are 3 ft. square, and deep enough to take in only the three upper ducts of the conduit.

After deciding on the streets to be used, the number of ducts, and the locations of manholes and service boxes as far as it is possible to locate the two latter from the surface of the street (for usually street paving covers a number of obstacles to conduit work), street maps may be got out,

WEIGHTS PER FOOT OF CAST-IRON PIPE OF VARIOUS THICKNESSES AND DIAMETERS.

Inside Diam. of Pipe in in.	THICKNESS OF METAL (HEAVY TYPE DENOTES STANDARD WEIGHT OF PIPE FOR 100 POUNDS PRESSURE)																		Weight of standard flanges for 100 lbs. pressure.					
	5/16 in.	3/8 in.	7/16 in.	1/2 in.	9/16 in.	5/8 in.	11/16 in.	3/4 in.	13/16 in.	7/8 in.	15/16 in.	1 in.	1 1/16 in.	1 1/8 in.	1 3/16 in.	1 1/4 in.	1 5/16 in.	1 3/8 in.	Inside Diam. of Pipe	Outside Diam. of Flanges	Thickness of Flanges	Number of Bolts	Size of Bolts	Diam. of Bolt Circle	Weight of Each Flange
	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.	Lbs.							
3	10.26	12.55	14.91	17.35	19.87	22.46	25.14	27.89	30.71	33.62	.....	.....	.....	.....	.....	.....	.....	.....	3	7½	¾	4	⅝	6	7.25
4	.....	16.27	19.25	22.31	25.45	28.66	31.95	35.30	38.77	42.29	.....	.....	.....	.....	.....	.....	.....	.....	4	9	¾	4	⅝	7½	12.50
5	.....	19.81	23.39	27.04	30.76	34.57	38.44	42.40	46.43	50.54	54.73	.....	.....	.....	.....	.....	.....	.....	5	10	¾	8	⅝	8½	14.37
6	.....	.....	27.91	32.22	36.60	41.05	45.59	50.19	54.88	59.64	64.49	69.40	.....	.....	.....	.....	.....	.....	6	11	1	8	⅝	9½	17.40
7	.....	.....	31.99	36.87	41.83	46.86	51.96	57.15	62.41	67.75	73.17	78.66	.....	.....	.....	.....	.....	.....	7	12½	1 1/16	8	⅝	10¾	23.33
8	.....	.....	36.60	42.14	47.75	53.45	59.22	65.07	70.99	77.00	83.08	89.23	.....	.....	.....	.....	.....	.....	8	13½	1⅛	8	⅝	11¾	27.24
9	.....	.....	.....	46.70	52.89	59.15	65.49	71.90	78.39	84.96	91.61	98.33	105.13	.....	.....	.....	.....	.....	9	15	1⅛	12	⅝	13¼	33.17
10	.....	.....	.....	52.05	58.91	65.84	72.85	79.94	87.10	94.35	101.67	109.07	116.54	124.09	.....	.....	.....	.....	10	16	1 3/16	12	⅝	14¼	41.03
12	.....	.....	.....	.....	70.06	78.23	86.48	94.81	103.21	111.70	120.25	128.89	137.61	146.40	.....	.....	.....	.....	12	19	1¼	12	⅝	17	55.54
14	.....	.....	.....	.....	81.22	90.63	100.12	109.68	119.33	129.05	138.85	148.72	158.68	168.71	178.81	.....	.....	.....	14	21	1⅜	12	1	18¾	68.98
15	.....	.....	.....	.....	.....	96.02	106.07	116.15	126.35	136.59	146.92	157.34	167.83	178.41	189.02	.....	.....	.....	15	22½	1⅜	16	1	20	76.03
16	.....	.....	.....	.....	.....	103.02	113.75	124.55	135.44	146.40	157.44	168.56	179.75	191.02	202.37	.....	.....	.....	16	23½	1 7/16	16	1	21¼	87.20
18	.....	.....	.....	.....	.....	.....	127.38	139.43	151.55	163.75	176.03	188.38	200.81	213.32	225.91	238.57	.....	.....	18	25	1½	16	1⅛	22¾	96.29
20	.....	.....	.....	.....	.....	.....	141.02	154.30	167.66	181.10	194.62	208.21	221.89	235.63	249.46	263.37	.....	.....	20	27½	1 11/16	20	1⅛	25	123.09
22	.....	.....	.....	.....	.....	.....	153.37	167.79	182.25	196.81	211.46	226.17	240.94	255.81	270.77	285.80	.....	.....	22	29½	1 13/16	20	1¼	27¼	143.34
24	.....	.....	.....	.....	.....	.....	.....	184.05	199.89	215.80	231.80	247.87	264.02	280.25	296.56	312.94	329.40	.....	24	32	1½	20	1⅜	29½	169 00
26	.....	.....	.....	.....	.....	.....	.....	197.28	214.21	231.24	248.33	265.48	282.73	300.07	316.46	334.94	352.50	.....	26	34½	1 15/16	24	1⅜	31¾	197.43
28	.....	.....	.....	.....	.....	.....	.....	212.02	230.21	248.45	266.76	285.17	303.64	322.20	340.82	359.54	378.32	.....	28	36½	1 15/16	28	1⅜	34	217.47
30	.....	.....	.....	.....	.....	.....	.....	228.66	248.23	267.86	287.57	307.36	327.23	347.18	367.20	387.30	407.48	.....	30	38½	1 15/16	28	1½	36	238 63
36	.....	.....	.....	.....	.....	.....	.....	273.28	296.56	319.91	343.34	366.85	390.44	414.10	437.84	461.66	485.56	.....	36	45½	2	32	1½	42¾	326.40
40	.....	.....	.....	.....	.....	.....	.....	.....	.....	354.61	380.53	406.52	432.56	458.72	484.95	511.24	537.62	559.64	40	50	2⅛	36	1⅝	48	391.67
48	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	.....	485.83	516.86	547.94	579.13	610.40	641.71	667.31	48	57½	2½	40	1⅝	54¾	461.70

as shown in Fig. 3 (original scale 50 ft. to 1 in.), and the estimate on the cost of the work made up. Making an estimate on work of this kind is somewhat difficult, for the engineer does not know what he may find under the paving, or what kind of weather will prevail. These two features are very important factors in the cost of the work.

It is a good plan for the company's engineer to take the City Engineer into his confidence as either he or some of his employees know much about locations of fire cisterns, water and gas pipes, sewers, etc. In putting in the work men-

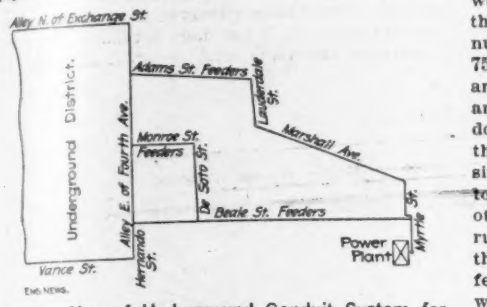


Fig. 1. Map of Underground Conduit System for Feeders at Memphis, Tenn.

F. G. Proutt, Chief Engineer, Memphis Consolidated Gas & Electric Co.

tioned in this article, we found that submitting all our plans to the City Engineer before beginning any work, benefited us, not only in the good feeling this created, but in the fund of information to be obtained from him and his assistants, and in the very timely suggestions he often had to offer for overcoming obstacles that might be encountered in carrying on this work. If you find out from the City Engineer which of his assistants has been longest connected with the sewer department, this individual can usually tell you not only how to drain the manholes but practically where all the pipes in the street are located.

The very first thing to be decided, however, in making up the estimate, is the style of duct to be used. There are several kinds on the market; wood, cement, paper, vitrified clay, etc. While all are no doubt good, we believe that the vitrified clay is the best. We also think that multi-duct section is better than single-duct section, and in our work used six-duct section and three-duct section only. We adopted the practice of entirely surrounding the conduit with concrete. As this is only a protection against picks of workmen in making other excavations, and also to prevent dirt from washing in at the joints, a really good concrete is not necessary; still it should not be too poor. We used Portland cement concrete, with crushed rock (washed gravel may be used with good results), making the mixture 1 part cement, 4 of sand and 8 of crushed rock. In any ordinary ground this makes a thoroughly satisfactory mixture. The thickness of concrete should be 3 ins. all around the conduit. In the above mixture, the finished volume of concrete will be practically the same as the volume of stone; the sand and cement simply filling the interstices between the stones, and as Portland cement has practically 4 cu. ft. to the barrel, the cost of concrete may be readily estimated.

The next thing will be the amount of dirt to be handled, which can be easily estimated. The price of handling will depend largely on the disposition to be made of the dirt, and the relative amount of backfilling as compared with total excavations. In laying multiple-duct conduits it is better to make a narrow deep ditch and put the sections one on top of the other, rather than to have a shallow wide ditch. The time of laying will be much less in the former case, less dirt will have to be moved, less pavement will be torn up, and less concrete used. The top of the concrete should not be less than 30 ins. below surface of paving. This gives room for water and gas services, and puts a good cushion of earth above the ducts, so that there is no chance of very heavy vehicles crushing the duct material.

In an 18-duct run made up of multiple ducts of six sections each, each section would measure about 9 x 13 ins., and laying three sections one above the other would require a ditch as follows: 3 ins. for lower concrete, 3 ins. for upper concrete, 27 ins. for ducts, 30 ins. for earth and paving above ducts, or a total depth of 5¼ ft. The width will be 13 ins. for duct, and 3 ins. on each side for concrete, making 19 ins. From this the amount of earth to be excavated can be estimated and also the amount to be backfilled.

In the work covered by this article, manholes are located about 500 ft. apart, and the service boxes have three-duct openings, so that at any future time the secondary system may be changed from overhead to underground without any further duct work, except the running of lateral mains from the duct system to the premises of customers. The number of ducts provided for is approximately 75% more than the number at present required, and should provide for all future requirements for an indefinite period. In fact, when the business done in the present underground district reaches the limit of the ducts provided, it would be desirable to run a third set of ducts from the plant to the business portion of the city down some other street. This is because it is undesirable to run more than 25 cables over any one route, as the cost of distribution from the point where the feeders strike the main underground system would, after going a certain distance, be greater than the cost of running a new set of feeders over another street, and striking nearer the desired center of distribution. As the city increases in population, of course, the amount of current used in the underground district will increase and the district will be enlarged from time to time; where pole lines have to be rebuilt, it will be a better investment to discontinue them and build an underground service.

The distances are about as follows: 1, Power plant to 4th Alley and Adams St., 6,000 ft. (18 ducts); 2, Power plant to Beale and De Soto S s., 4,000 ft. (27 ducts); 3, De Soto St. and alley north of Monroe St. to 4th Alley, 2,000 ft. (18 ducts); 4, Beale St., from De Soto to Hernando Sts., 500 ft. (18 ducts). In all there will be about 13,150 ft.

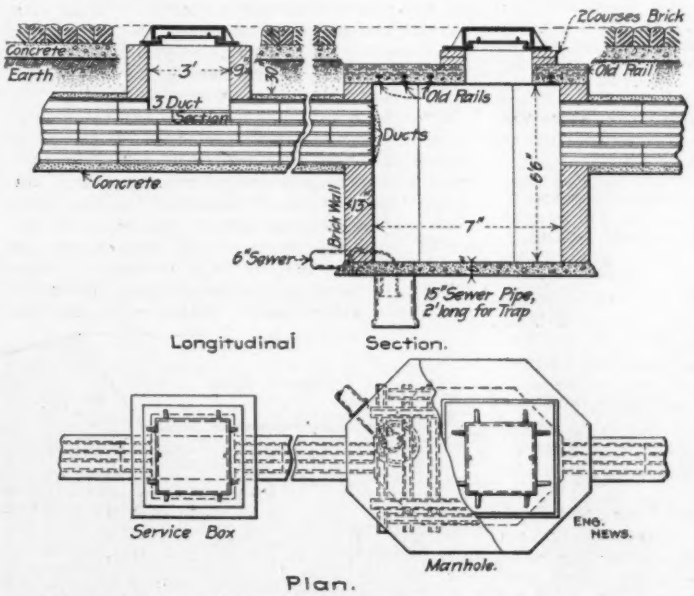


FIG. 2. STANDARD MANHOLE AND SERVICE BOX ON CONDUIT SYSTEM.

of conduit, with 264,300 ft. of duct, and a summary of the cost of duct work will be as follows:

Trenching and moving earth, at $1 per cu. yd. (including backfilling, tamping and hauling away surplus)	$4,550
Concreting at $4 per cu. yd.	4,180
Duct material, at 6¼ cts. per duct-ft.	16,519
Duct laying, ½ ct. per duct-ft.	1,321
40 manholes, $250 each (this allows $100 for the drain from manholes)	10,000
40 service boxes, $35 each	1,400
Tearing out concrete in streets	400
Replacing concrete	700
Repaving	500
City inspection of the work	600
Engineering	2,000
Incidental expenses (lighting, watching, etc.)	1,000
Tools, lumber, etc.	1,000
Total	$44,170
Add 5% for contingencies	2,208
Total cost of duct work.	$46,378

The total estimated cost of work was $46,378, and as the work was done by the day we will next take up the method of doing it and compare the actual cost of the different materials with the estimated cost. In this way we can see the errors made in estimating, and bring out especially the fact that in estimating work of this kind, too much care cannot be taken in going into the special features of the work, such as crossing streams and subways, and making connections at the station end. Crossing streams of water is very expensive, as a special structure has often to be built for this work, an example of which is described later. In laying out a duct system it is well to avoid as far as possible all streets having concrete and brick or asphalt pavements, streets passing over railway tracks or streams of water, and streets having car tracks. Of course some of each must be encountered, but often parallel street or alleys will answer for main lines of duct systems at a great saving in cost of construction.

After making up the estimate on the work and having it passed by the proper authorities, the next thing to consider is whether the work shall be put in by contract or day work. The cost of engineering on the work during construction will be practically the same (as far as the company is cerned) in either case. Cost of inspection will be more for the company if work is done by contract, as inspectors will have to be employed to see that contractors are complying with specifications, while if work is done by the day the foremen take the place of the inspectors and are actually pushing the work along. Materials and labor can, as a rule, be purchased as cheaply by the company as by a contractor, but if a company cannot get the proper men to handle the work, it should by all means let the work by contract, as day work would be a losing proposition. If such men can be had, the chances are that the company can do the work as cheaply as a contractor and save the contractor's profits. As a rule, however, contract work will be the cheaper method. In our case, the writer had had a somewhat extensive experience in conduit work and had several men who had been employed in this class of work for several years, so the proper thing seemed to be to carry out the work by the day and this plan was pursued. All the items of cost quoted hereafter are exact, and obtained from practical experience with the work.

CONSTRUCTION WORK.

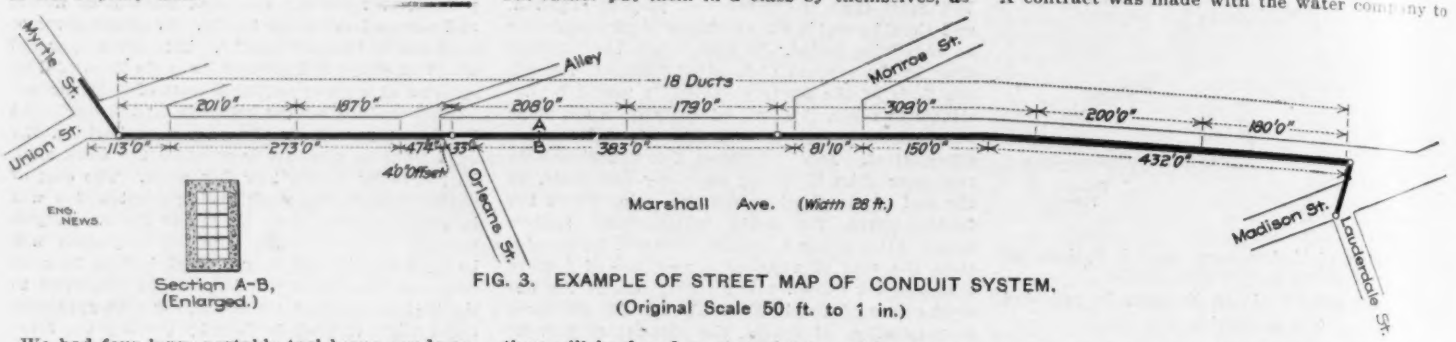
In carrying on the work the first thing to have is a good organization of labor. There should be: 1, a general foreman; 2, a foreman of pipe layers; 3, a foreman of concrete mixing gang; 4, a foreman in charge of digging for manholes and service boxes; 5, a foreman in charge of back-filling and hauling away dirt. Also, a timekeeper; this position is a very important one, and the man filling it should be young, well educated and thoroughly reliable.

The general foreman should not be an engineer, but rather a man who has worked a gang laying sewer, water or gas pipe, or had experience in ditching, as this man must take the lead and keep the trench opened up. We were fortunate enough to obtain a man of this kind, at $18 per week. We have found that a good foreman always has a big following of laborers who will go on the work with him, provided, of course, the work will last several months. This foreman was hired a month before work began, and was kept busy with one or two laborers building boxes in which to mix mortar for the brick masons, building platforms on which to dump concrete, making gages for the ditchers (which are simply pieces of lath cut to

the width of trench), making small platforms to put across the trench at people's gates, and wide bridges to put across the trench at street intersections. All these things must be got ready before work is begun.

Then an outfit of tools must be made up. This, in our case, consisted of the following:

2 dozen wheelbarrows.	2 claw hammers.
7 dozen round-point shovels.	3 hand saws.
	1 hatchet.
1 dozen square-point shovels.	1 bundle oakum.
	2 "dromedary" concrete mixers.
7 dozen picks.	
30 iron tamping bars.	4 or 5 crow bars.
36 red lanterns.	4 chisel bars, 3 ft.
2 axes.	6 sledge hammers.
6 or 8 balls of cotton cord.	2,000 ft. common lumber, 1 × 6 ins., 14 ft.
25 lbs. 10-penny nails.	
25 lbs. 20-penny nails.	



Section A-B, (Enlarged.)

FIG. 3. EXAMPLE OF STREET MAP OF CONDUIT SYSTEM.
(Original Scale 50 ft. to 1 in.)

We had four large portable tool boxes, made as shown in Fig. 4, and a small portable cement house, Fig. 5. The tools also included 24 galvanized pails, and 50 ft. of 2½-in. hose with a 1-in. faucet in the end.

While the foreman was making up the platforms, contracts were made for stone, sand, cement, water, etc. The ducts had, of course, been ordered some time previous, or in fact as soon as it was decided to put in the work. The following letter was sent out to each of several manufacturers:

We are now in the market for 45,000 ft. of 6-duct section, making 270,000 duct-ft. and 4,666 ft. of 3-duct section, making 14,000 duct-ft., or 284,000 duct-ft. of underground conduit material. In addition to this we will require 200 pieces of 6-duct section 6 ins. long; 200 pieces 12 ins. long, and 200 pieces 18 ins. long. These pieces are for turning curves and finishing out at manholes.

We will require the necessary dowel pins to use in lining up the ducts. We also require that the 284,000 duct-ft. of material be made up in 3-ft. lengths.

Kindly quote us price as soon as possible on this material delivered f. o. b. cars Memphis, your company to stand all breakage that occurs in transit. The material must be thoroughly vitrified and glazed, and be entirely free of scales or projections on the inside of the ducts, and all crooked pieces will not be accepted. In other words, we want to purchase first-class material.

It will be noted that the letter calls for 284,000 duct-ft. in place of 264,300 as estimated, the excess being intended to cover breakages and also for some short lateral runs that it was thought it might be advisable to put in with the balance of the work. Even should some duct material be left over, it would be good stock, not suffering from exposure to weather even if carried on hand for a year or two.

It will be noted also that ducts were to be delivered f. o. b. Memphis, and all crooked, broken or badly glazed pieces rejected. When the contract for ducts was finally placed (at 5½ cts. per duct-ft.) and the material began to arrive, it was found that each car would contain from one to a dozen pieces, out of which corners would be broken. The engineer then agreed to accept all such broken pieces at half the price of a sound piece. That is a 6-duct section 3 ft. long would contain 18 duct-ft. and be worth 99 cts.; but if a corner was broken off the section, it would be accepted at 49½ cts. All such damaged pieces were used in the top run of ducts in the trench and a piece of sheet zinc placed over the hole caused by the corner being broken off. Duct and zinc were all finally covered with concrete. This method of accepting damaged pieces was considered satisfactory by all parties concerned.

All 6-duct sections were supposed to be 3 ft. long and all 3-duct sections, 2 ft. long. Owing to uneven shrinkage in the clay, however, the manufacturers are unable to make the material run exactly to length; some pieces would come 34 ins. long, some 35 ins. and some 36 ins. As each car arrived it was unloaded, each piece examined and gaged, and shortage noted. The shipper was then notified of value of shortage and breakage and he issued a credit bill covering the amount. These credits run all the way from $1 to $10.50 for shortage, and from 50 cts. to $20 per car for breakage, so that they were well worth considering. The 3-duct sections were bought 2 ft. long in place of 3 ft., as the manufacturers claimed they could turn out much straighter material in 3-duct section of the shorter length; as the price per duct-ft. was the same in either case, we conceded the point.

The engineer should not reject crooked pieces, but rather put them in a class by themselves, as they will be found most useful in making curves where it is necessary to run the duct line over or under obstacles that may be encountered in the trench. There is, however, one class of duct that should be rejected always, and that is the section which measures 9 × 13 ins. at one end and 10 × 14 ins at the other. If accepted and hauled to the ditch to be laid, much trouble will result from attempting to make its big end coincide with the small end of a neighboring section.

The specifications above quoted will be found entirely satisfactory to the purchaser, as it gives him the authority to reject unfit material and the shipper stands breakage in transit. In all such transactions the engineer should be fair minded and be willing to pay a fair price for all material that can be utilized, thus making the shipper feel that he is at least being honestly dealt with.

It will be noted that the duct specifications call for a certain number of short pieces, 6, 12 and 18 ins. long. Agents selling ducts will frequently state that their ducts can be cut like cheese. But as short sections can be bought at the same price per duct-ft. as long sections, it pays infinitely better to buy a few short lengths than to find out from experience that cutting a vitrified clay duct is much the same as cutting a glass lamp chimney with a pair of shears. Following up the idea of buying pieces of any desired length, when all our duct laying had been completed, except evening up the ends in manholes, we ordered a lot of short sections, 1, 2, and 3 ins. long to do this. So that from beginning to end we decided to let the manufacturer do all cutting for us, and we simply butted the ends together and put in the dowel pins.

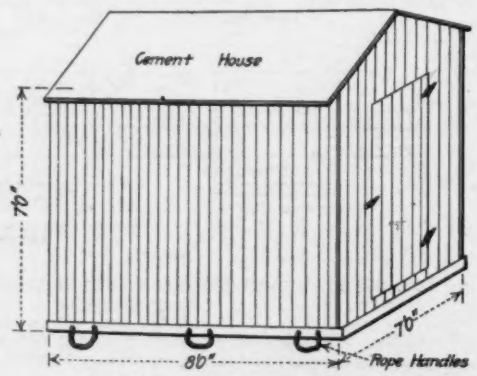


Fig. 5. Portable Cement House.

In work of this kind, all material should be purchased to be delivered right on the work if possible. We bought 1½-in. crushed stone (with the dust not sifted out), delivered on the work, at 10 cts. per 100 lbs., or from $2.50 to $3 per cu. yd., there being a difference in the weight of different limestones per cu. yd. Sand was bought in the same way at $1.25 per cu. yd. and "Diamond" Portland cement at $2.10 per bbl. delivered. Cast-iron tops for manholes and service boxes were bought at $1.90 per 100 lbs., each top weighing 1,150 lbs. Hard bricks delivered on the work cost us $7 to $7.50 per 1,000, and an arrangement was made with a firm of brick contractors to furnish us all the masons and laborers we needed at $6 and $2 each per day, respectively. Teams were engaged at $4 per day for two horses and driver, and $3 per day for one horse, cart and driver. Two or three private carts belonging to colored citizens were hired at $2.50 per day, including a driver. A contract was made with the water company to allow the use of all the water required on the work for $50, and then permission had to be obtained from the Fire Department to attach hose to the fire hydrants, as these hydrants belong to the city.

After all arrangements of this kind had been made, the other foremen were employed and work was begun. For a timekeeper we engaged a young man who had worked for us a year and then entered Purdue University to take an electrical engineering course. He was then on his vacation, the work commencing July 6, 1903. The foreman of concrete mixers was also a former employee, and a fellow student of the timekeeper. The wages of these men were $2 and $2.25 per day respectively. The foreman who looked after manhole and service box excavations received $2 per day, and the same was paid to a foreman who looked after hauling dirt, while one of our regular men at $16.50 per week saw that conduit material was always on the ground and kept up with little details of all kinds.

Everything being ready, a Monday morning was chosen for the start. The general foreman lined up his men, and the timekeeper took their names. The wagon loaded up the tool boxes and placed them along the proposed trench line, and the cement house was placed at a point about 300 ft. from the starting of the trench. The sand, cement and crushed rock men were notified to deliver material, and work was begun. As soon as the trench was well started, a manhole excavation was commenced, and when this was done the brick masons began work. Duct sections were hauled out from the storage yard (which was located on a railway siding and rented for this purpose at $10 per month) on two-horse platform wagons, two of them being kept busy all the time. The duct was piled along beside the trench in a continuous line, at such a distance from it as would leave room for a wheelbarrow to pass between the trench and the ducts, with openings about 50 ft. apart in the pile of ducts to allow the wheelbarrows to pass through.

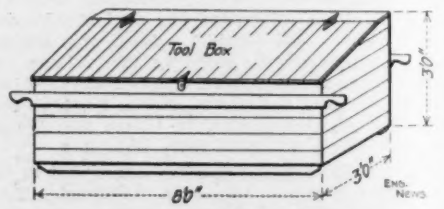


Fig. 4. Portable Tool Box.

Fig. 6 shows a concrete mixer being charged. The mixer was of the "dromedary" type, in which the concrete is mixed during transportation. It held about ⅝-cu. yd. and was charged as follows:

A little sand is first put in, then 1¼ sacks of cement (which was all delivered in sacks), then more sand, then the stone. A test mixture was first made of 1 cement, 4 sand and 8 stone. After this, the sand was gaged by a mark on the mixer, the cement by the bag, and stone enough added to fill the mixer. The door was then closed and the mixer driven about 150 ft. to the water plug and water enough put in to make a good wet concrete. This amount of water was from 3 to 8 pails, depending on the amount of dust in the stone. It was then driven to the dumping place and dumped as shown in Fig. 7; at these places

Fig. 6. Loading.

Fig. 7. Dumping.

CONCRETE MIXING AND HANDLING ON MEMPHIS CONDUIT SYSTEM.

planks were located close to the trench and close to where ducts were being laid.

The dumping boards were platforms 6 × 12 ft., made of two thicknesses of 1-in. pine, one thickness laid lengthwise and one crosswise. At each outside edge of the platform was a piece of timber 2 × 4 ins., and rope handles, so that the platform could be picked up and carried along as the work moved ahead. Two platforms were placed end to end, making a platform 24 ft. long, which gave ample room to dump material. On unpaved streets platforms are very essential for good work. The mixers required two mules each and cost us $200 per mixer, f. o. b. Washington. They are especially suited for conduit work, saving a great deal of labor in wheeling by barrows for long distances.

The ditchers worked close together in the trench. A line was stretched 4 ft. from the curb as a rule, and the trench run outside of this line, each ditcher having a gage 19 ins. long (the width of the trench) to work by.

In laying ducts, the following method was followed: First, the 3-in. concrete bottom was put in; then the boss duct man and his helper got in the ditch and laid the lower run. Two men on the bank would hand down the sections by means of a rope run through one of the holes. As soon as the first runs had been started a few lengths, the second run would begin and so with the third and fourth and fifth, taking four men for each run or layer. The pipe used had dowel pin holes on the outside or around the periphery of the duct, which is the most convenient place for them as far as the duct layers are concerned. In a 6-duct section there were six dowel holes on each end. We only used 3 pins, however, in each end, one at each side and one on top.

The joint was made as follows: a piece of cheap domestic canvas 5 ins. wide and 5 ft. long was laid on the bottom of the ditch before the duct was put in position, then when the duct sections were in place there would be a piece lying directly under the joint. A boy followed each set of layers and wrapped the canvas up around the joint, overlapping the end and painting the lap with black asphaltum. This makes a first-class joint, as the canvas is only to serve the purpose of keeping the concrete out of ducts, and the cheaper this purpose can be accomplished, the better. As about 30,000 of these pieces of canvas were used on the work, a method had to be devised for cutting them. The canvas was purchased in bolts, costing about 5 cts. per yd. A rough table was made with a saw-cut in it 5 ins. from one edge, and at this edge was a strip against which to push the bolt of cloth. A large butcher knife was then run through the saw cut and cloth, cutting off a piece 5 ins. wide and the length of the bolt. This piece was wound on a reel whose circumference was 5 ft. and a cut through the cloth at the circumference made pieces 5 ft. long. This method was original, simple and thoroughly satisfactory.

In an 18-duct run the method of opening three top ducts into the service box was to put in a piece of 3-duct section at the box in place of 6-duct section. This made the lower three ducts pass through and left the upper three open.

The only skilled labor employed was for laying brick. The brick masons would lay up the walls of the manholes to the proper height, then move to the next hole. The gang excavating manholes would then return to the hole left by the brick masons and put up a temporary wood ceiling, laying on it (and projecting over the walls) some pieces of old rail; they then put a box where the cover was to be and left it. The concrete men would then put on 10 ins. of concrete and the hole was complete, ready for the cast-iron cover. This concrete reinforced by old rails makes a first-class top. The rails cost about $5 for each manhole, 5 × 7 ft., and the concrete about the same amount. The manholes, Fig. 2, are not square but octagonal in shape.

As it is the intention to use paper insulated cables almost entirely in these ducts, and as from

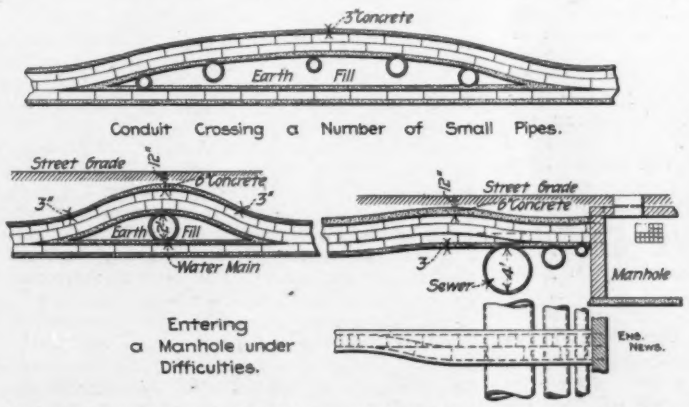


FIG. 8. EXAMPLES OF DIFFICULT WORK AT OBSTRUCTIONS TO CONDUIT SYSTEM AT MEMPHIS, TENN.

past experience we have found that paper insulation will not stand sharp bends like rubber covered cables, we decided to make the manholes of such shape that sharp bends could not be made in the cables. In building manholes it is well to have a standard size, but this standard cannot always be adhered to. In other words, manholes must be made to suit conditions and almost any shape will do, but by all means avoid making them too small. A manhole should not be less than 5 × 7 ft. inside, and every manhole should be drained. We had one case where a manhole drain cost $300, but the average cost was less than $100. We would, however, have put in the drains had they cost $300 each.

In carrying on this work it is well to remember that the main point of consideration is laying ducts. Therefore let the instructions to foremen morning, noon and night be to "lay ducts." Another thing to remember on quitting at night is that next day it may be raining; therefore, prepare for the morrow. Do not put conduit in 400 or 500 ft. of trench and leave it with no concrete around it. Every bit of pipe laid during the day should be covered with concrete before it is left at night. Following up this idea we had a number of times to work concrete men until 8 p. m. to get the work covered. For this overtime we paid them time and a half. The consequence was that we never had to pull out any work in the morning, and we lost less than $100 during the whole work on account of bad weather.

Another thing to keep in mind is that seven or eight men can excavate and build manholes, but it requires 75 or 100 men to trench and lay ducts; therefore, do not excavate for manholes faster than the masons can build them. Dig the trench straight through where the manhole is to be; lay ducts up to it on one side, then leave a space for the manhole and start laying ducts again on the other side. Always start all the duct runs even; that is, if there are four tiers of ducts, see that in starting, the ends all even up so that they will be flush with the manhole wall when it is built. But in ending the run at the next manhole opening, do not waste time trying to get the ends even, get them within 6 ins. or so and let them stand. Have the masons, in building around these uneven ends, leave the hole slack so that short pieces may be inserted after all other work has been cleared up. Do not let masons do any duct fitting at all. A couple of men can fill in short pieces wherever needed in a few days at the windup of the work, while they would delay all the pipe layers, bank men and concrete men in trying to do so when the whole work is being carried on.

It would be well to have ducts run in perfectly straight lines and without dips or pockets, but this in practice cannot be done. Perhaps the very first day the engineer will be confronted by the general foreman with the information that a 24-in. water main crosses the trench and the top of the main is only 36 ins. below the grade of the street. The engineer cannot sit down and make elaborate calculations as to the evil results of a dip in the duct line. While he is doing that, men are standing idle. He must do the best thing that can be done under the circumstances. If running an 18-duct section, put 12 ducts over the pipe and 6 under, and put 6 ins. of concrete on top of the ducts in place of 3 ins. Also arrange the ducts as shown in Fig. 8 to save concrete. If the earth is well tamped it will not settle after the concrete has set.

Again, you may strike a lot of pipes, say 4 ft. from the surface and right close to a proposed manhole. In our case we had the following problem: A large brick culvert crossed the ditch about 4 ft. from the surface, also two sewers and a water pipe. Two rows of the ducts were carried over the pipes, etc., and after crossing the obstructions, put back in the regular form of run, three in a tier. Fig. 8 shows work of this kind.

In another case, Fig. 9, a bayou had to be crossed under a bridge. This bridge was an old one and liable to be pulled down and replaced by a new one, so the duct line must be put over the bayou in such a way that the removal of the bridge would not interfere with the ducts. The method adopted was as shown: Four 10-in. channels 50 ft. long were purchased and made up into a truss with side lattice bars. A manhole was built at one side of the bridge and a pier built at one end of the manhole and as part of it, the manhole foundation being 12 ins. of concrete reinforced with old rails. A pier of concrete 6 × 8 × 4 ft. thick was built on the other

side of the bridge and the truss rested on these two piers, entirely independent of the bridge structure. The ducts were then laid through the truss and ducts and truss enclosed in concrete, making practically a reinforced concrete beam. The object of enclosing the truss in concrete was to prevent the steel from rusting.

Fig. 10 shows a case where we had about 20 sewer pipes crossing the ditch in 200 ft. The same means were employed in running the ducts as with the large water main. That is, one run of ducts was put under the sewers and covered on all sides with concrete, then earth tamped into

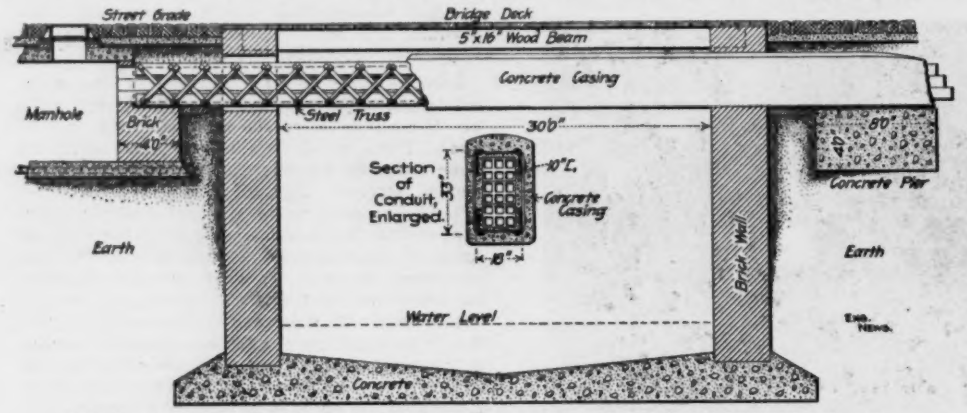


FIG. 9. CONDUIT CONSTRUCTION ACROSS THE BAYOU GAYOSO; MEMPHIS, TENN.

the top of the sewer pipes, then a new concrete bottom put in 3 ins. thick and the balance of ducts put in and concrete around.

It is advisable to have a report each night of the trench feet and duct feet put in for the day. At the end of each week, when the pay roll is made up, the duct feet laid for the week should also be made up and the price of labor per duct-ft. calculated. The greatest amount of work done by us in any one day was 15,156 duct feet in 703 ft. of trench. All our trenches were 19 ins. wide, and when 18 ducts were put in the depth was 5¼ ft., while when 27 ducts were put in, the depth was 6¼ ft. The 27-ducts run was made up of 4 multiples of 6 each and one multiple of 3, making 5 sections of ducts in a tier.

COST OF WORK.

Our pay roll on the work for all purposes for the 18 weeks was from $36 to $1,279, and with $350 for preliminary work, putting in sewer traps, evening up ends of duct lines in manholes, etc., the total cost for all labor was $11,525. Of the amount, $1,424 was for labor on excavation, building manholes and service boxes; $2,356 for labor on sewer work, and $7,745 for trenching, mixing concrete, laying ducts, backfilling and hauling away surplus dirt. As the amount of duct laid was 251,991 duct-ft., the cost for labor was 3.07 cts. per duct-ft. The total cost of cement used was $2,196, or practically 1,046 barrels. Of this amount there was used for brickwork in manholes, 160 barrels ($336); for brickwork in service boxes, 12 barrels ($25,20); and for jointing drain pipe, 5 barrels ($10.50). This leaves $1,824 as the cost of cement used in concrete.

The total amount of sand used cost $776.54 for 635 cu. yds., or an average cost of $1.22 per yd. Of this sand, 74 yds. were used for manhole and service-box brickwork, leaving 561 yds. for concrete work, the cost of which was $684.42. The total amount of broken stone used was 2,713,500 lbs., costing $2,713.50, so that the total cost of all concrete work was $1,824 for cement, $684.42 for sand, and $2,713.50 for stone, making $5,221.92 for practically 1,000 cu. yds. of concrete, or $5.22 per cu. yd.

Of the concrete 118 yds. were used for manhole bottoms and tops and service-box bottoms. The cost of this was $615.96, leaving $4,605.96 as the cost of concrete used around conduits, or a cost of 1.83 cets. per duct-ft. The total number of bricks used for building manholes and service boxes was 118,000, which cost $871. There were 32 manholes and 48 service boxes built, manholes averaging 3,200 bricks each, and service boxes 325 bricks each. Service boxes cost complete $30.15 each, or $1,447 for 48 boxes. The average cost of manholes was $115, without sewer, and the average cost per sewer was $76 for labor and $10 for sewer pipe, making $86. The average cost of a complete manhole was thus $201. The average length of each sewer was 170 ft., 6-in. pipe being used, and the total amount paid for sewer pipe being $309.50. Only 31 sewers were run, as one manhole was built beside an old manhole and the old drain sufficed for both.

The total cost for tools of all kinds and keeping same in repair was practically $800. The cost of city inspection was $195. The cost of engineering was about $1,000. The amount paid the city for repairing streets was $1,000. The cost of various odds and ends, such as cotton cloth for covering joints, asphaltum for painting same, dowel pins for keeping ducts in alinement, unloading cars and various other incidentals was $2,230.71. The steel truss for crossing the Bayou Gayoso cost $700, and the sum of $600 was paid for new sidewalks where duct lines ran under sidewalk, making a total cost of $41,234.56 for all the work done, as shown by our books. A summary of the work as completed shows the following:

Length of trench, ft.	No. of ducts.	Length of duct, ft.
216.2	60	12,975
3,415.0	27	92,205
7,226.5	18	137,277
324.0	24	7,776
293.0	6	1,758
11,474.7		251,991

The estimated amount of duct-ft. to be laid was 264,300, while the actual amount laid was only 251,991 duct-ft. The difference was a short run that was not put in, owing to the city having laid an asphalt pavement on the desired street, and it was thought advisable to leave out this small run until such time as its construction would be required.

The total amount of duct material bought, however, cost $15,564, which at 5½ cts. per ft. would represent 282,987 ft. After completing the work we had in stock 28,506 duct-ft. of unbroken material or material undamaged in any way. This added to the amount put in accounts for 280,497 duct-ft. and the difference of 2,490 ft. represents the loss by breakage, this loss being less than 1% of the total amount purchased. The loss through breakage can only be kept so low by buying the ducts to be delivered f. o. b. at the place where they will be used, and arranging for payment for damaged sections, as stated earlier in this article.

Summing up the total results of our experience in doing our own construction, we find as follows:

Estimated Cost of Work.		Actual Cost of Work.	
Trenching	$4,550	Labor on ducts	$7,745
Concrete	4,180	Concrete for ducts	4,606
Duct material	16,518	48 service boxes	1 447
Duct laying	1,321	32 manholes	3,680
40 manholes	10,000	31 manhole drains	2,666
40 service boxes	1,400	Duct material	15,564
Tearing out concrete	400	Tools	800
Replacing concrete in streets	700	City inspection	195
Repaving	500	Repaving streets	1,000
City inspection	600	Steel truss over bayou	700
Engineering	1,000	New sidewalks	600
Incidentals	1,000	Incidentals	2,231
Tools, lumber, etc.	1,000		
Contingencies, 5%	2,208	Total	$41,234
Total	$46,378		

In regard to the incidentals, $2,231 seems a large amount. But it cost about $7.50 to unload and inspect a car of ducts and there were not less than 50 cars; there were used over 40,000 dowel pins at ½-ct. each, and about five barrels of black asphaltum paint at about $30 per barrel; we also paid $50 for city water and about $100 to plumbers for mending broken water pipes (where in trenching the men would occasionally drive a pick through the lead service pipe). It will be seen, therefore, that the amount would soon be made up, as the items enumerated are only a few of the many items embraced in this account.

Leaving out the cost of manholes, service boxes and manhole drains, it will be noticed that the cost per duct-ft. of the work complete was $33,441.56 divided by 251,991 duct-ft., or 13.27 cts. Had we put in the entire number of duct feet estimated and also the number of manholes and services estimated, our total cost for ducts would have been 264,300 × 13.27 or $35,072 for ducts, $1,206 for service boxes, and $8,040 for manholes, making a total of $44,318, plus $1,000 for engineering, which item has not yet been charged in the account. Thus at the prices paid, had the original estimate been adhered to exactly, the cost would have been $45,318 and the estimate $46,378.

One reason for changing the number of manholes was that in certain locations before the street was opened, is appeared as if two manholes would be required, one on each side of a culvert, for instance, while on opening the street one manhole would be found sufficient, with perhaps an extra service box.

One item not estimated at all was the steel truss over Bayou Gayoso. This was an oversight, but (fortunately) enough manholes were left out to more than pay for its construction. Another item left out was the iron pipe required from the switchboard floor at the power plant to the first manhole. The cost of this will be about $800, but will not have to be considered until the cables are to be installed. We have, however, a large item of credit that has also not been considered in the foregoing figures. There is 28,506 duct-ft. of new duct material in stock, the value of which is $1,567, and this credit will almost balance the engineering expenses which have not yet been charged up, and also the cost of running in the station ends of iron pipe, which were not estimated on.

In conclusion, I would say that throughout the entire work we were exceedingly fortunate as to

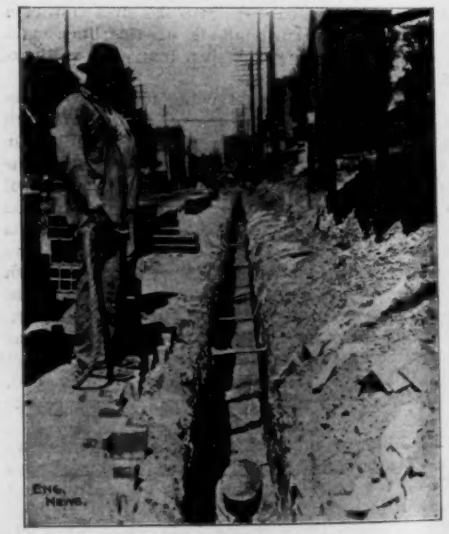


Fig. 10. Trench Work on Conduit System.

weather, labor, and material to be excavated; and these are the three items which may either make or break a contractor. If all are in his favor, he will come out ahead, while if all are against him, he is almost sure in work of this kind to come out in the hole. We have not enough outside data on conduit work to know whether our costs are either abnormally small or abnormally large, but we do know that the figures given are absolutely correct and therefore offer them to the readers of the Engineering News for consideration.